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NATIONAL DAM SAFETY PROGRAM. LAKE COHOON DAM (INVENTORY NUMBER --ETC(U)
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Name Of Dam: LAKE COHOON DAM
Location: CITY OF SUFFOLK
Inventory Number: VA. 12301

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED FOR

NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

DEWARD M. MARTIN & ASSOCIATES
WILLIAMSBURG, VIRGINIA
AUGUST 1979

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

LAKE COHOON DAM
CITY OF SUFFOLK, VIRGINIA
(Formerly Nansemond County)
INVENTORY NO. VA 12301

LOWER JAMES RIVER BASIN

Name of Dam : Lake Cohoon Dam
Location : City of Suffolk (formerly Nansemond County)
Inventory Number: VA 12301

PHASE I I[illegible]CTION REPORT

National Dam Safety Program

Prepared for

NORFOLK DISTRICT CORPS OF ENGINEERS
803 Front Street
Norfolk, Virginia 23510

by

Deward M. Martin & Associates, Inc.
August 1979

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of the Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the design flood should not be interpreted as necessarily posing a highly inadequate condition. The design flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

BRIEF ASSESSMENT OF DAM

Name of Dam:	Lake Cohoon Dam
State:	Virginia
County:	City of Suffolk (Formerly Nansemond County)
USGS Quad Sheet:	Windsor, Virginia
Stream:	Nansemond River
Date of Inspection:	May 7, 1979

Lake Cohoon Dam is located in the City of Suffolk 2.15 miles south of U S Route 460 at Providence Church. The dam is an earth embankment about 900 feet in length from abutment to abutment. The dam is 35 feet high from the top to the toe. The top of the dam is 12 feet wide. The dam is classified as intermediate size and significant hazard. The dam is owned by the City of Portsmouth, Department of Public Utilities. The purpose of the dam is water storage for the treatment plant and recreation, for the City of Portsmouth. The dam has a concrete spillway that is V-shaped and has a crest length of 300 feet. The spillway consists of three 10-foot wide steps with each having a height of 5 feet.

Based on criteria established by the Department of the Army, Office of the Chief of Engineers (OCE), the spillway is rated as inadequate. The spillway will only pass 26 percent of the PMF, while the Spillway Design Flood (PMF) will overtop the dam for 18 hours and reach a maximum of 2.3 feet over the top of the dam, with an average critical velocity of 8.7 feet per second. Since the spillway will not pass 1/2 of the SDF, the dam is assessed as "unsafe-non-emergency" in accordance with guidelines presented by the Corps of Engineers. The water level readings in the observation wells on the crest of the dam show that the core wall is apparently not functioning and that the stability of the dam is questionable.

It is recommended within 6 months that the following work be accomplished:

a. The phreatic surface within the dam should be surveyed and verified.

b. A more detailed study of the downstream flood plain and of the Spillway Design Flood appropriate to this dam should be conducted. Remedial measures to be considered include modification to the dam, spillway, flood plain, and/or any other method of eliminating the danger imposed by the project.

c. An annual maintenance and inspection program should be initiated to help detect and control problems that may occur.

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Date SEP 17 1979

LAKE COHOON DAM

Top of Dam

Spillway - Front View

LAKE COHOON DAM

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SECTION 1

PROJECT INFORMATION

1.1 General:

1.1.1 Authority: Public Law 92-367, 8 August 1972 authorized the Secretary of the Army, through the Corps of Engineers to initiate a national program of safety inspections of dams through the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I Inspection according to the Recommended Guidelines for Safety Inspection of Dams (Appendix V, Reference 1). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Lake Cohoon Dam is an earth embankment dam 900 feet in length from abutment to abutment. The dam is 35 feet high from the top at elevation 33 to the toe at elevation -2. The top of the dam is 12 feet wide. The upstream slope of the dam is 2.5 (H): 1(V) and 2(H) : 1(V) on the downstream side of the embankment. The upstream slope has stones uniformly placed 2 to 3 feet above normal pool elevation. There is a concrete cut of wall from elevation -15 to elevation 32 along the center of the dam. The cut of wall is 12 inches wide at elevation 32.

The dam includes a V-shaped concrete spillway which has a crest length of 300 feet . The open end of the Vee is 121 feet wide with the closed end extending upstream from the toe of the dam. The spillway consists of three 10-foot wide steps with each having a height of 5 feet. The crest of the spillway is at elevation 28.0.

There is a gate house accessible by a pedestrian bridge from the top of the dam. Within the gate house there are manually operated controls for three 24-inch valves and two 30-inch sluice gates.

1.2.2 Location: Lake Cohoon is located 2.15 miles south of U S Route 460 at Providence Church along State Route 604 about 0.5 miles south of the intersection with State Route 638.

1.2.3 Size Classification: The dam has a storage capacity of 9,400 acre feet. Therefore it is classified as intermediate according to the storage capacity.

1.2.4 Hazard Classification: The dam is immediately upstream from three buildings with an estimated population of 10 people, and is therefore given a significant hazard classification in accordance with section 2.1.2 of the Recommended Guidelines for Safety Inspection of Dams, published by the Department of the Army, office of the Chief of Engineers. The hazard classification used to categorize the dams is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: Lake Cohoon is owned by the City of Portsmouth, Department of Public Utilities.

1.2.6 Purpose of Dam: Lake Cohoon is used for water storage for the water treatment plant and recreation by the City of Portsmouth.

1.2.7 Design and Construction History: The lake Cohoon Dam was constructed in 1912. In 1919 the War Department Construction Division increased the height by 13 feet, raising it to elevation 33 at the top of the dam. At the same time, a V-shaped concrete spillway was constructed with the crest at elevation 28 and a crest length of 300 feet.

1.2.8 Normal Operational Procedures: When pool level is above spillway crest, the water automatically flows into Lake Meade. When the pool level is below spillway crest, the water is directed by manual operation of the valves through the 30-inch diameter pipes to Lake Meade.

1.3 Pertinent Data:

1.3.1 Drainage Area: The dam controls a drainage area of 33.3 square miles.

1.3.2 Discharge at Dam Site:

Maximum flood - unknown

Two 30-inch outlet pipes
pool level at spillway crest . . . . . . 216 cfs.

Spillway
pool level at top of dam . . . . 10,400 cfs.

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are shown in the following table:

Table 1.1 DAM AND RESERVOIR DATA

Item	Elevation feet m.s.l.	Reservoir Area, acres	Reservoir Capacity Acre, feet	Reservoir Capacity Watershed, inches	Length miles
Top of Dam	33.0	850	9,400	5.3	5.0
Spillway crest	28.0	500	6,025	3.4	4.3
Streambed at the toe of the dam	-2±	---	----	---	---

SECTION 2

ENGINEERING DATA

2.1 Design: The available information consists of plans prepared under the direction of the War Department Office of Construction Quartermaster in 1919. The plans indicate there is a concrete core wall from the marl line (upper limit of the Yorktown Formation) to one foot below the top of the dam.

2.1.1 Operational Record: The dam is used as a storage facility for the City of Portsmouth, Virginia. It was constructed in 1912 and altered in 1919 to raise the elevation of the dam from elevation 20 to elevation 33. (See Appendix IV, Section C, paragraph 2B, for details.) The level of the reservoir is normally regulated by water flowing over the spillway crest into Lake Meade. The water level can also be regulated by opening one or more of the gate valves located in the gate house (see section 4 this report.)

*2.2.1 Geologic Setting of the Dam: The dam is located in the Coastal Plain physiographic province and is underlain by the Yorktown Formation of Miocene geologic age. The Yorktown consists generally of preconsolidated marine sand, clay and broken shell material. West of the Nansemond River the upper 5 to 10 feet of the Yorktown Formation is usually orange to yellow in color before grading into its characteristic gray to green color. Surrounding hilltops in the immediate dam area are usually capped with the Sedley Formation of Pliocene geologic age. The Sedley is composed of fine sand and silty sand with thin layers of silty clay. This formation averages about 10 feet in thickness and was not encountered in the test borings.

*2.2.2 Available Geotechnical Data: Geologic information obtained in conjunction with the original investigation of the dam was not available. However, a geotechnical investigation of the dam was conducted by Schnabel Engineering Associates in 1978. The report of this study is included in Appendix IV. Seven soil test borings were drilled in conjunction with the investigation, four along the crest and three along the downstream toe of the dam. Two observation wells were installed on the crest of the dam in conjunction with the soil test borings. An observation well was installed on the upstream and downstream side of the core wall (cutoff wall), to measure the effectiveness of the wall.

*2.2.3 Dam Foundation: The dam is founded on a layer of alluvial soils consisting of silty and clayey sands. This is designated as "Stratum B" in the 1978 Report by J. K. Timmons and Associates and Schnabel Engineering (see Plates 2, 3 and 4 of Appendix IV.) No strength tests were performed on these soils, however, total and effective friction angles of 35° and total and effective cohesions of 0 were assigned to the stratum. These alluvial soils are underlain by clayey sands, clayey silts and silty clays of the Yorktown Formation (Stratum C.) No strength tests were performed in this stratum.

*Information provided by Law Engineering Associates of Virginia.

*2.2.4 Embankment: The embankment consists of two materials, referred to as Stratum A and A1 on the subsurface profiles in Appendix IV. Stratum A, from the ground surface to a depth of 14 to 39 feet, consists of silty to clayey coarse sand, of a very loose to firm density. Laboratory testing indicated a total friction angle of 13° and a total cohesion of 400 psf for the Stratum A soils. The tested effective angle of internal friction for Stratum A was 37° with 0 cohesion. The insitu desnity of Stratum A ranged from 80 to 89% of the standard Proctor Maximum dry density (based on two compaction tests.)

Stratum A1, interbedded with Stratum A to a depth of 4 to 20 feet, consists of silty clayey madium sand and silty clay of a soft to very stiff consistency. The strength of this stratum was not determined by testing, however, total and effective friction angles of 4° and 37°, respectively, and total and effective cohesions of 700 psf and 0, respectively, were estimated for Stratum A1.

Stability analyses were conducted by Schnabel Engineering Associates in conjunction with the subsurface investigation performed in 1978. The loading conditions of both steady state seepage and sudden drawdown of the reservoir were considered for both the upstream and downstream embankment slopes. The seepage line for steady state seepage conditions was developed from readings taken at the observation wells along the crest of the dam. The phreatic surface used for the section used by Schnabel in their analysis is shown on Plate 2, Appendix IV.

The results of the stability analysis are as follows:

Surface	Case	Loading Condition	Factor of Safety	Required Min. Factor of Safety
Upstream	I	Sudden drawdown of Reservoir	1.3	1.2
	III	Steady Seepage	1.9	1.5
Downstream	I	Sudden drawdown of Reservoir	1.2	1.2
	III	Steady Seepage	1.5	1.5

2.3 Evaluation: Borings and laboratory data have been obtained by Schnabel Engineering Associates in 1978. This data does not adequately define the strength of the foundation soils, however, since no strength tests were performed. The assumed values appear reasonable and adequate enough to accurtely evaluate the strength, however, laboratory tests should be performed. In addition, the density of the embankment soils was not adequately determined since only two compaction tests were performed.

*Information provided by Law Engineering Associates of Virginia.

SECTION 3

VISUAL INSPECTION

3.1 Findings:

3.1.1 General: The results of the May 7, 1979 inspection are recorded in Appendix III. At the time of inspection the pool elevation was 28.2 m.s.l., about 2.5 inches above normal pool elevation. There was about 2 - 3 inches of water flowing over the spillway. There was an inspection report available, done by J. K. Timmons and Associates in 1978.

3.1.2 Dam: There is no obvious horizontal or vertical misalignment in the dam. Minor undulations were observed in the downstream slope, however, no cracking, settlement or bulging was observed. A path was worn into the downstream slope approximately five feet up from the pool. Minor erosion of the surface soils has resulted along the path. The placed stones on the upstream slope needs to be relaid where some small depressions have occured. There is no obvious sloughing on the visible slopes. There was no cracking or erosion noticeable at the abutments. Approximately 10 tree stumps were observed on the downstream slope of the embankment to the left of the spillway. The trees appeared to have been cut recently. Water level readings were taken in two observation wells located on the crest of the dam. According to the geotechnical report enclosed in Appendix IV these wells are located on either side of the concrete core wall. In addition, two hand auger probes were made along the downstream slope of the dam. Approximate water levels were obtained from these probes. The water level readings obtained from the wells and probes indicated the core wall was not functioning. A typical cross section of the dam with an approximate line of seepage based on the water level readings taken is shown on Plate 7 in Appendix 1.

3.1.3 Appurtenant Structures: The valves are in operating condition according to the representative of the owner, however, the walkway and gate house are in need of repairs.

3.1.4 Abutments: The abutment walls show white stains, spalling, and actual seepage. At the left abutment wall, about 4 feet downstream from the end of the slope wall, there is a joint showing seepage of less than 1 GPM accumulation of mud and small roots. The abutment walls showed signs of spalling and reinforcing bars were visible in some locations.

3.1.5 Spillway: The spillway, which had water flowing over the crest at the time of inspection, had some spalling of the concrete at the leading edge of the spillway steps and at the joints.

Soil and water was seeping through the left spillway wall, at less than 1 GPM. Approximately 20 feet downstream from this seepage location, water was seeping from the embankment soils, over the concrete retaining wall and into the spillway. The approximate location of these two areas is shown on plate 6 in appendix I.

3.1.6 Instrumentation: During the 1978 investigation by Schnabel Engineering Associates (Appendix IV), four observation wells were installed. Two of the well were installed on the crest of the dam in conjunction with the soil test borings. Wells were also installed on the upstream and downstream sides of the core wall to measure its effectiveness. Measurements in the wells were taken at the same time of the inspection and are included in Appendix I as Plate 7.

3.1.7 Reservoir Area: The surrounding area is densly wooded. Land adjacent to the left side of the spillway had recently been cleared of small trees and brush. There is still considerable debris (branches, stumps, etc.) covering the ground surface. On the crest and upper part of the downstream slope at the southernmost section of the dam, several large tree stumps were present. These trees had recently been cut down.

Tan fines were clouding an approximately ten foot square area of the downstream pool adjacent to the shoreline. The approximate location of this area is directly down slope of observation Well B-3 (Plate 7, Appendix I .) The clouded area indicates the presence of fines which may indicate piping.

Water with soil fines was seeping through the left spillway wall at a rate of less than 1 gallon per minute (gpm.) Approximately 20 feet downstream from this seepage location, water was seeping from the embankment soils, over the concrete retaining wall and into the spillway at a rate of less than 1 gpm. The approximate locations of these two areas is shown on Plate No. 6 in Appendix 1.

3.2 Evaluation: The visual inspection revealed a number of deficiencies which need further study and/or remedial action. Information from Law Engineering Associates in conjunction with this report indicates that the core wall may not be functioning properly (see Plate 7, Appendix I.) It is recommended that the owner, over the next 3 months, monitor the two wells on either side of the core wall to determine whether or not the wall is functioning properly. If the water level on the downstream side of the wall remains near the level on the upstream side, the owner should secure the services of a professional engineer to conduct a more thorough investigation to assess the condition of the wall and to recommend appropriate remedial action. It is further recommended that the owners' engineer investigate and evaluate the potential damage, through piping, that may be done by the tree stumps and root systems mentioned in 3.1.2 above. The loose stones on the upstream face of the embankment should be relaid and the depressions filled. A program should be initiated to repair the spalling and deterioration of the concrete surfaces on the spillway and abutment walls. At the same time, the areas of exposed reinforcing steel should be repaired. The gatehouse and the walkway leading to it are also in need of repair.

SECTION 4

OPERATIONAL PROCEDURE

4.1 Procedure: The normal storage pool is at elevation 28.0 which is the crest of the spillway. When the water elevation is above 28.0, the water automatically flows over the spillway into Lake Meade. When the pool elevation is below the spillway crest, water can be discharged into Lake Meade by one or both 30-inch pipes running from the base of the gate house in Lake Cohoon to Lake Meade. These 30-inch pipes have sluice gates located at the gate house. The gate house is divided in half for each 30-inch pipe. One side of the gate house is filled by opening one, two, or three of the 24-inch gate valves. The other side is filled by the removal of the double timber stop logs. The valves or double timber stop logs are opened or removed to help supply the water needs for the City of Portsmouth during high water demands. The Water Department of Portsmouth makes the decision when and how much water is to be discharged from Lake Cohoon to Lake Meade.

4.2 Maintenance of Dam: A complete routine maintenance program has not been established for the Cohoon Dam, although periodic maintenance has occured. Daily tasks of maintenance such as mowing grass and greasing valves are done by the City of Portsmouth, Department of Utilities. Other maintenance, such as repairs to the gate house, removal of trees or stumps, and repairs to the concrete spillway are done by contract.

4.3 Maintenance of Operating Facilities: The operating facilities consist of the valves to the pipes through the embankment and the spillway.

4.4 Warning Systems: There is no warning procedure or evacuation plan established by the owner to follow in case of an emergency.

4.5 Evaluation: There is presently a program under consideration by the owner and his engineer which will repair the gate house bridge, riser house, and clean the sluice gates to enhance operations. The 24-inch gate valves are in good operating condition. An annual maintenance and inspection program should be initiated to help detect and control problems that may occur. The operating procedures are adequate.

SECTION 5

HYDRAULIC/HYDROLOGIC DATA

5.1 Design: Construction plans from R. Kenneth Weeks, dated January 21, 1919.

5.2 Hydrologic Records: None were available.

5.3 Flood Experience: No records available.

5.4 Flood Potential: The PMF and 1/2 PMF were routed through the reservoir. Hydraulic routing and data were furnished by the Corps of Engineers.

5.5 Reservoir Regulation: Pertinent dam and reservoir data are shown in Table 1.1.

Water is passed from Lake Cohoon to Lake Meade during high water demand periods. Two 30-inch pipelines from a gate house in the Lake Cohoon run through the dam to the Lake Meade. Water also flows past the dam over the spillway in the event water in the Lake Cohoon rises above elevation 28.0.

Rating curves were generated by the Corps of Engineers.

5.6 Overtopping Potential: The probable rise of the reservoir and other pertinent information on reservoir performance is shown in Table 5.1.

Table 5.1 RESERVOIR PERFORMANCE

Item	Normal flow	Hydrograph 1/2 PMF	Hydrograph PMF (c)
Peak flow c.f.s.			
Inflow	3	20,500	41,000
Outflow	--	19,500	39,700
Maximum elevation feet, m.s.l.		34.5	36.8
Spillway (elevation 28)			
Depth of flow, feet (a)		4.5	6.0
Velocity, fps (b)		12.0	13.8
Non-overflow Section (elevation 33)			
Depth of flow, feet (a)		1.0	2.3
Duration, hours		11	18
Velocity, fps (b)		5.7	8.7
Tailwater elevation, feet m.s.l.	12±	19.9±	24±

a. Critical depth

b. Velocity at critical depth

c. The PMF is an estimate of flood discharge that may be expected from the most severe combination of critical meteorolotic and hydrologic conditions that are reasonable possible in the region.

5.7 Reservoir Emptying Potential: Two 24-inch gate valves in the gate house with one at elevation 21.0 and another at elevation 1.0 are available for dewatering the reservoir. The valves will permit withdrawal of about 216 c.f.s. with the reservoir level at the crest of the spillway and essentially dewater the reservoir in about 28 days. With the downstream reservoir, Lake Meade, at the normal water elevation (12.0), Lake Cohoon can only be lowered to the same elevation.

5.8 Evaluation: Corps guidelines indicate the appropriate Spillway Design Flood (SDF) for an intermediate size and significant hazard dam is 1/2 PMF to PMF. Because of the risk involved, the PMF has been selected as the SDF. The spillway will pass 26 percent of the PMF. The SDF will overtop the dam by a maximum of 2.3 feet with a critical velocity of 8.7 fps and remain above the top of the dam about 18 hours.

SECTION 6

STRUCTURAL STABILITY

6.1 Foundation and Abutments: The dam is constructed on a thin layer of alluvial soils which is underlain by Yorktown deposits. The concrete core wall, located in the center of the dam extends from 1 foot below the top of the dam to the marl line and is founded in the Yorktown Formation. There was no obvious misalignment in the abutment walls although some signs of spalling and seepage were detected. No abutment drains were noted during the inspection

Stability calculations were performed by Schnabel Engineering Associates in 1978, using a combination of tests and assumed soil parameters. The results of the stability analyses are reported in Section 2.

6.2 Embankment: The soil for the earth embankment was placed in 6 to 8 inch layers and rolled between layers. It is not known whether density requirements were placed on the embankment material. The downstream slope is 2(H):1(V) and the upstream slope is 2.5(H):1(V). Stone riprap has been placed on the upstream slope and it shows signs of deterioration where stones have become dislodged. A number of trees have been cut on the downstream slope of the dam to the left of the spillway and seepage is noticeable on the spillway wall at this location. The combination of these two items make the stability of the embankment questionable, at least in this area. In addition, the report from Law Engineering indicates that the core wall may not be functioning properly (see Plate 7, Appendix 1.) The readings from the observation wells as shown on this drawing indicate that the water elevation on either side of the core wall is approximately the same.

6.2.1 Design Stability: The dam is located in Seismic Zone 1. Activity in this area is low and the possibility of damage due to an earthquake is considered to be negligible.

6.3 Evaluation: Visual observations leave some questions regarding the overall stability of the dam. The tree stumps, seepage and cloudy water at the downstream side of the embankment, to the left of the spillway, leave some doubt regarding the stability of the embankment. This area should be monitored regularly to detect further deterioration. Insufficient strength data was available to analytically evaluate the structural stability of the dam, however, the high water level on either side of the core wall indicates cause for concern. The water level in the test wells should be monitored regularly to see if this condition persists. Should the condition remain, it is recommended that the owner, at his own expense, secure the services of a professional engineer to conduct further investigations of the core wall and embankment to more accurately ascertain the dams stability.

SECTION 7

ASSESSMENT AND REMEDIAL MEASURES/RECOMMENDATIONS

*7.1 General Condition: From a geotechnical standpoint, the dam appears to be functioning well, with the exception of the observed seepage along the left abutment and the cloudy nature of the water downstream of observation well B-3 as discussed in Section 3. In addition, water level readings obtained during the inspection indicate the core wall is apparently not functioning properly.

*The available geotechnical engineering data is considered to be inadequate in some areas, primarily in regard to soil strength. Assumed parameters, for Stratum A1, B and C, have been used in the stability calculations in regard to strength. The estimated and tested strength parameters appear reasonable based upon previous experience with similar soil types and empirical correlations with standard penetration tests. However, estimated values are not considered adequate for dam studies, especially in light of the calculated safety factors being the minimum accepted values for certain loading conditions. Secondly, the stability calculations were based upon the core wall functioning properly which may not have been the case at the time of the inspection.

The visual inspection indicated the need for an upgraded maintenance program. The deterioration of the concrete and reinforcing steel in the spillway and abutments should be corrected to prevent further damage and the vegetation on the embankment should be cut regularly to prevent the growth of large trees and bushes which can cause damage to the dam. The extent of the damage caused by the trees on the lower side of the embankment to the left of the spillway is difficult to evaluate now and therefore should be monitored regularly to detect any changes or increased seepage. In addition, the water levels observed in the test wells on either side of the core wall seriously question its stability.

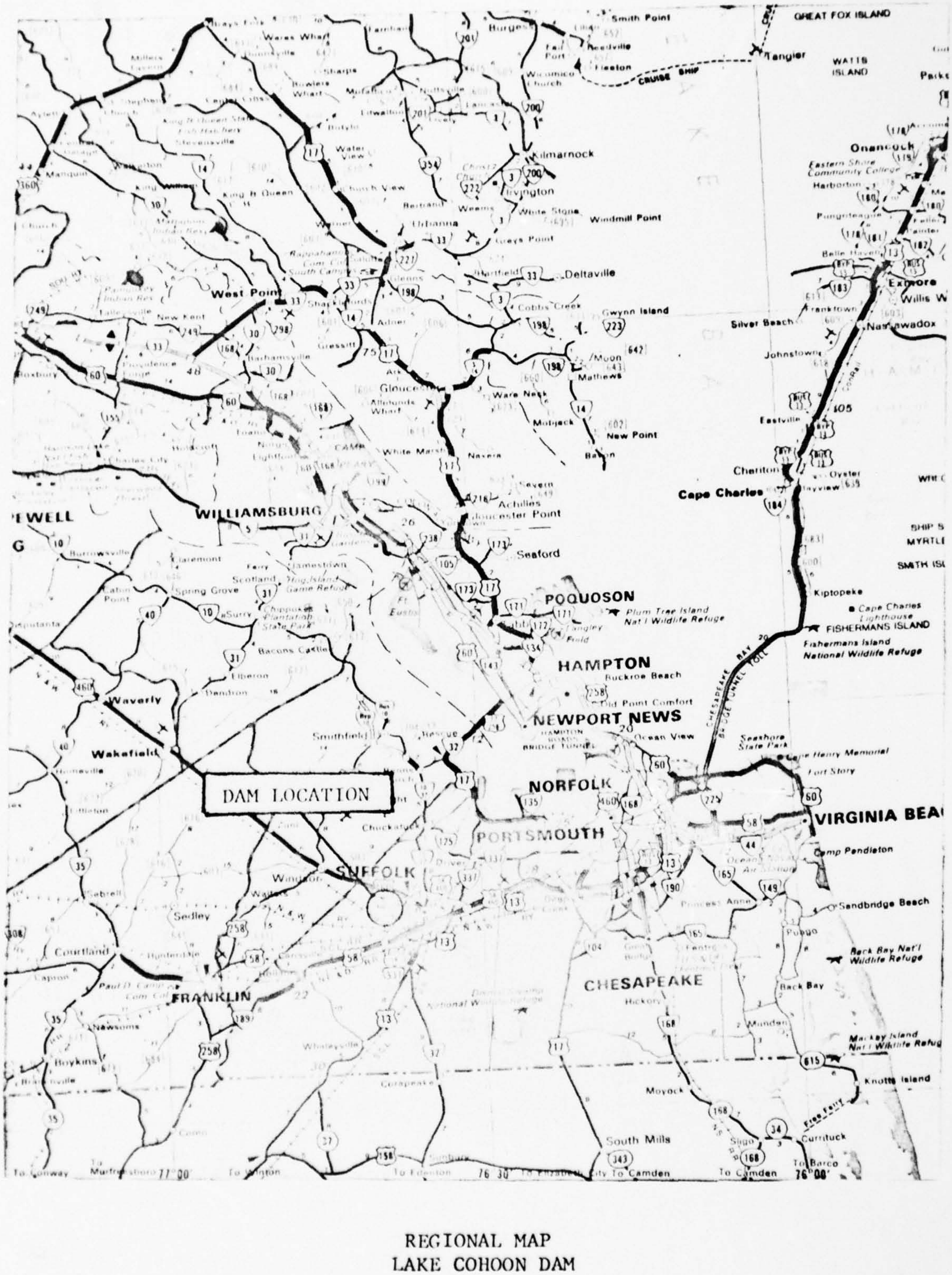
The spillway for the dam will pass 26% of the Spillway Design Flood (PMF) without overtopping the dam. The PMF will overtop the dam by 2.3 feet at a velocity of 8.7 feet per second. Since the spillway will not pass 1/2 of the SDF, the dam is assessed as "unsafe-non-emergency" in accordance with guidelines presented by the Corps of Engineers.

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening, and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

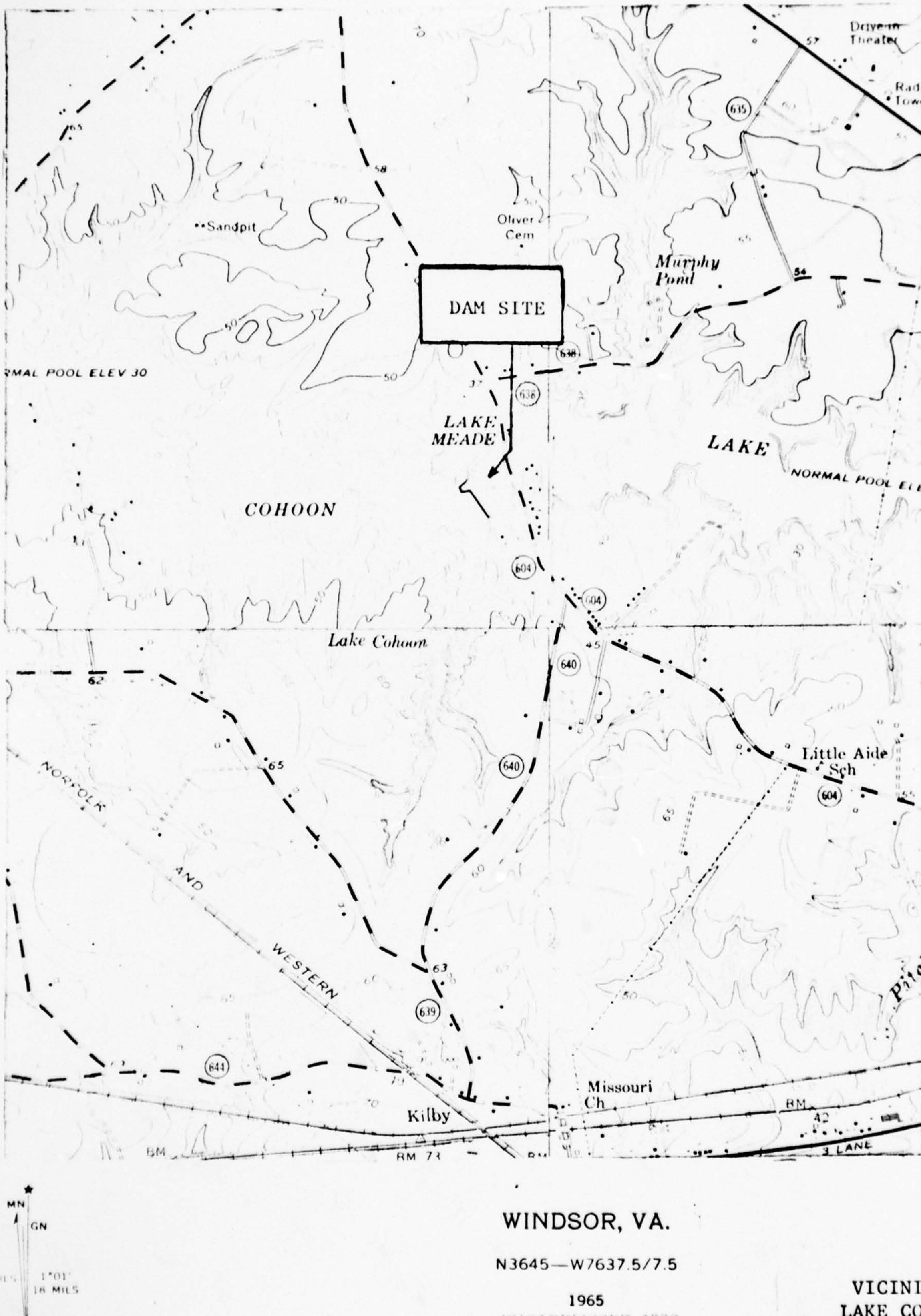
7.2 Recommended Remedial Measures: It is recommended that the owner, over the next 6 months, monitor, on a regular basis, the water elevations on either side of the core wall. If the water level on the downstream side of the wall remains at or near the height shown in Plate 7, Appendix I , the owner should secure the services of a registered professional engineer to perform a detailed inspection and stability analysis on the dam. He should also evaluate the potential that the tree stumps located along the dam have for creating high seepage gradients and subsidence upon decay. The owner should immediately take action to prevent further deterioration of the concrete surfaces and reinforcing steel. A regular maintenance and inspection program should be initiated to help detect and control problems that may occur and a warning system should be established for the dam.

APPENDIX I

MAPS AND DRAWINGS



REGIONAL MAP
LAKE COHOON DAM



WINDSOR, VA.

N3645—W7637.5/7.5

1965

PHOTOREVISED 1972

AMS 5657 I SW—SERIES V834

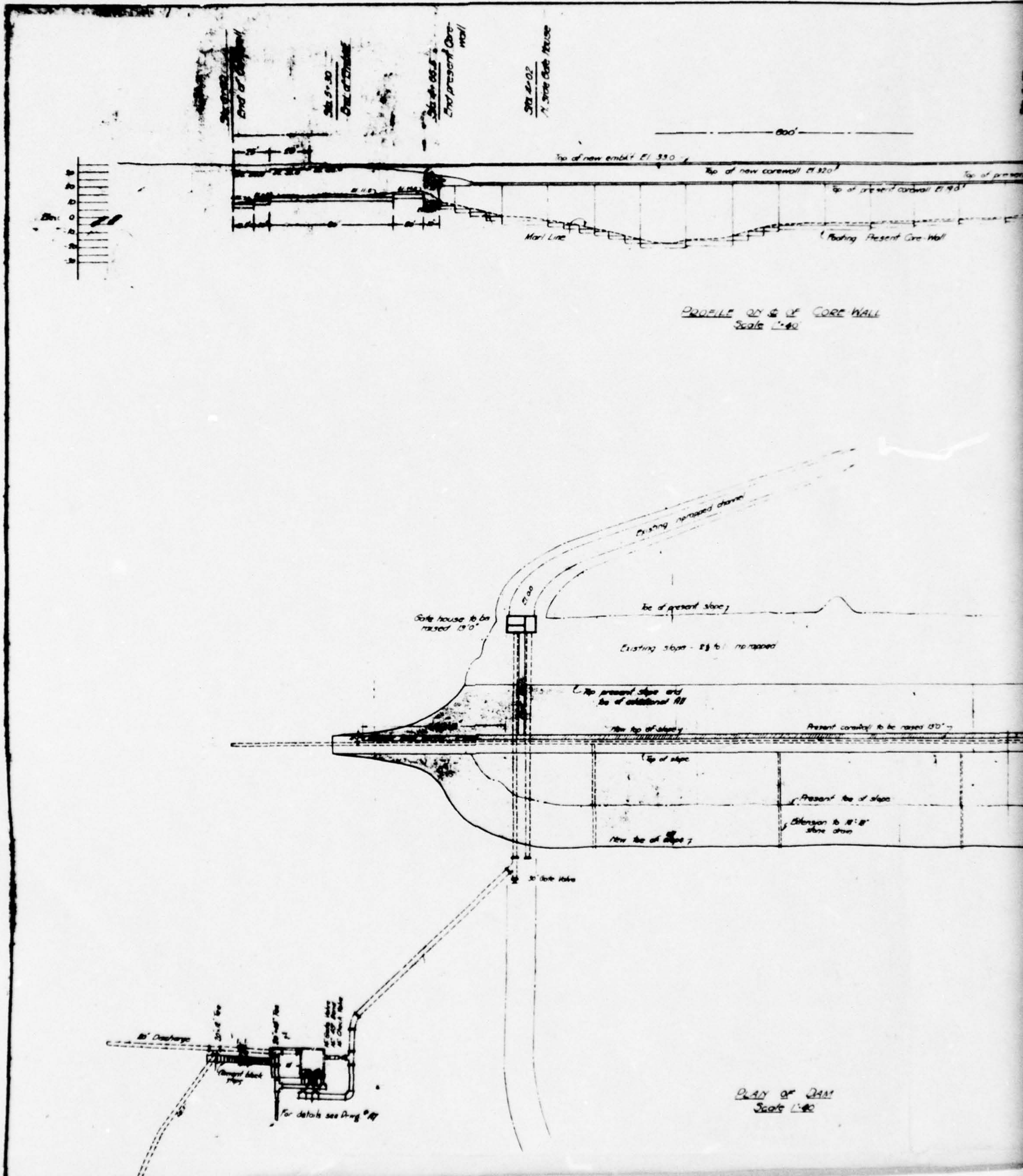
scale 1"=2000'

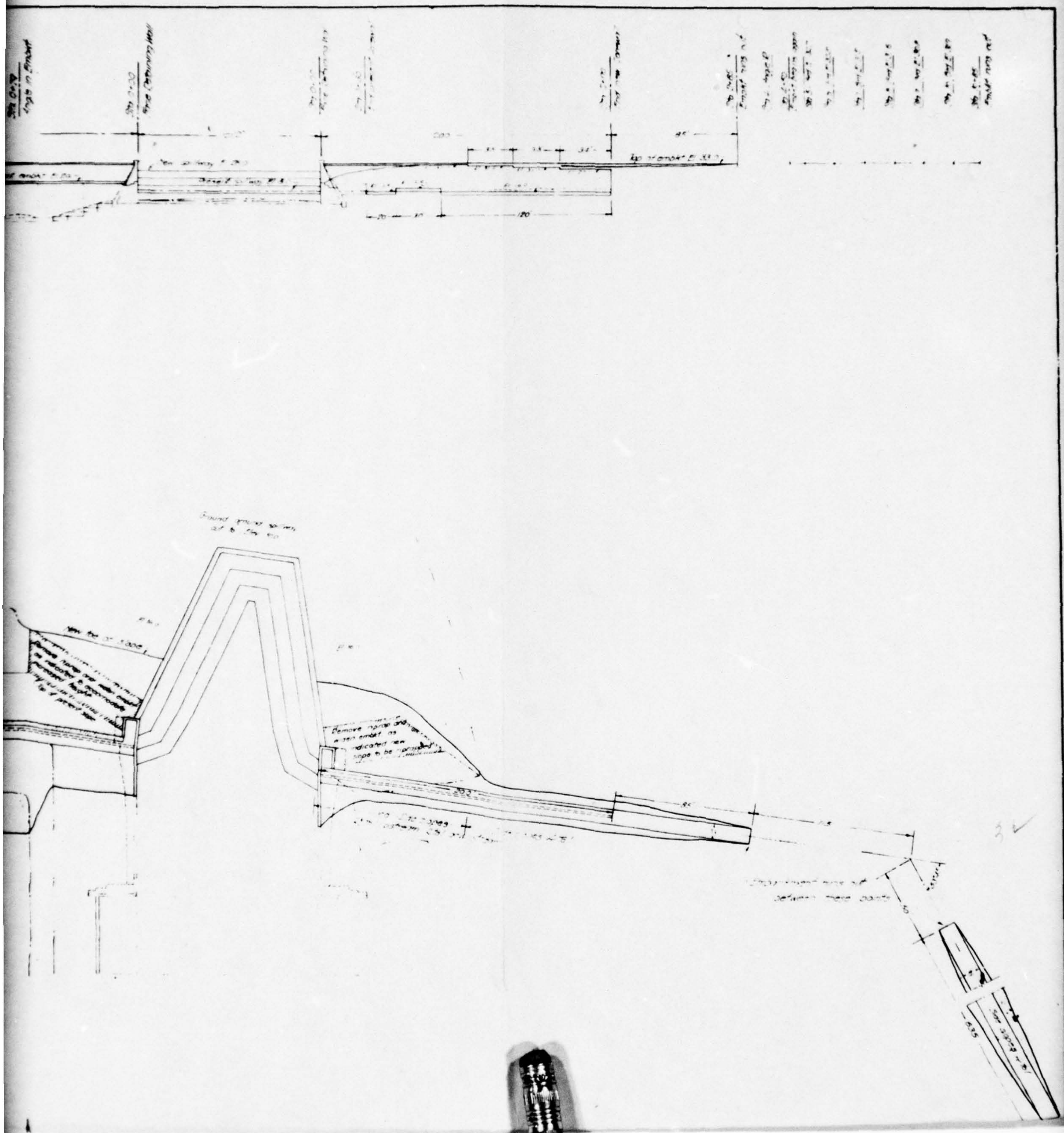
10' contours

VICINITY MAP

LAKE COHOON DAM

LAKE COHOON


Sta 9+30
End of Dike
Sta 2+02
N. Side Gate house
800'
Top of new embk't El 33.0
Top of new corewall El 32.0
Top of present corewall El 19.0
Elev.
Marl Line
Footing Present Core Wall
PROFILE ON ℄ OF CORE WALL
Scale 1"=40'
Existing riprapped channel
Gate house to be raised 13'0"
Toe of present slope
Existing slope - 2½ to 1 riprapped
℄ Top present slope and toe of additional fill
New top of slope
Present corewall to be raised 13'0"
Top of slope
Present toe of slope
Extension to 18" stone drain
New toe of slope
30" Gate Valve
20" Discharge
For details see Drwg # 17
PLAN OF DAM
Scale 1"=40'



3

1

REDUCED SCALE
HALF-SIZE PLAN

WAR DEPT. - CONSTRUCTION DIVISION.

PORTSMOUTH WATER DEVELOPMENT

— JOB - 208 —

PORTSMOUTH, VA.

OFFICE OF CONSTRUCTING QUARTERMASTER

Plan & Profile of Dam

SCALE 1" = 40'

APPROVED BY [illegible signature]

DATE 1-24-1918

REVISION APPROVED BY ________ DATE ____

________ DATE ____

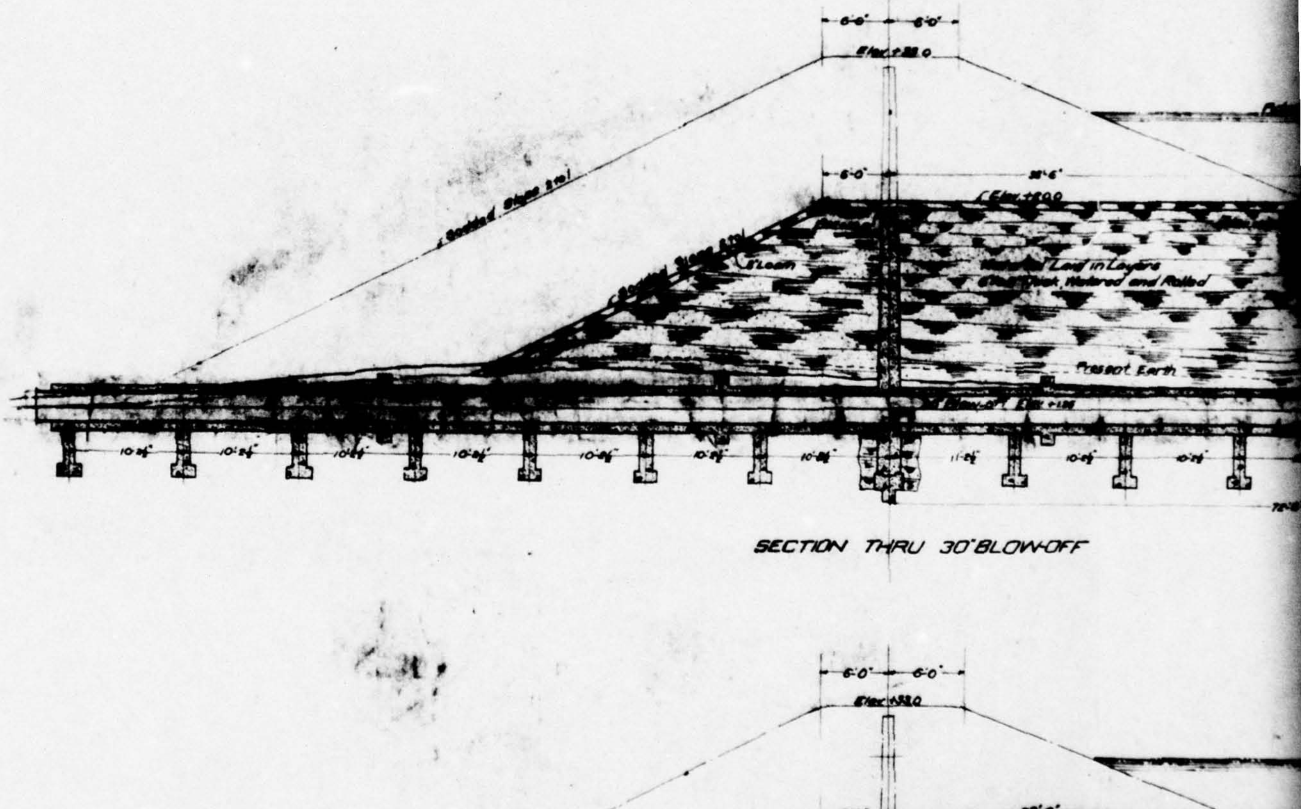
ULEN CONTRACTING CO. — ENGINEERS & CONTRACTORS

DESIGNED BY [illegible] | CHECKED BY [illegible] | TRACED BY [illegible] | DRAWING NO. [illegible]

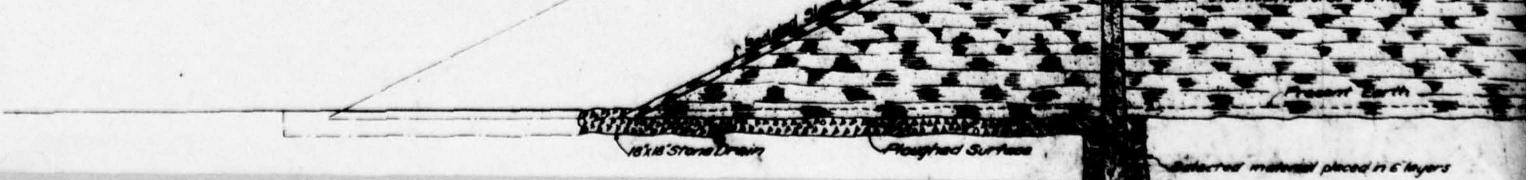
PLATE NO. 1

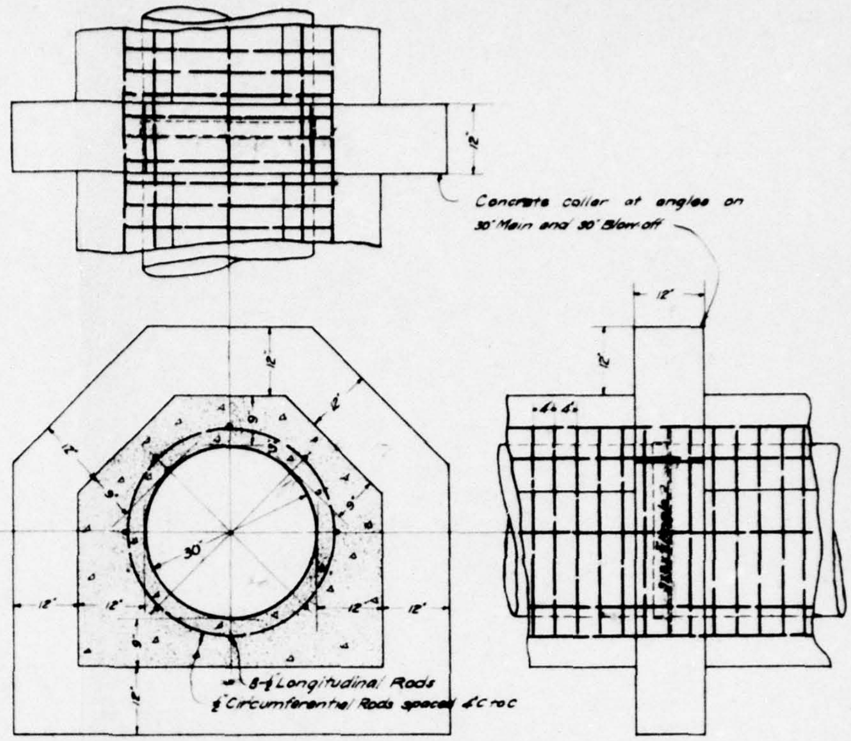
4

1

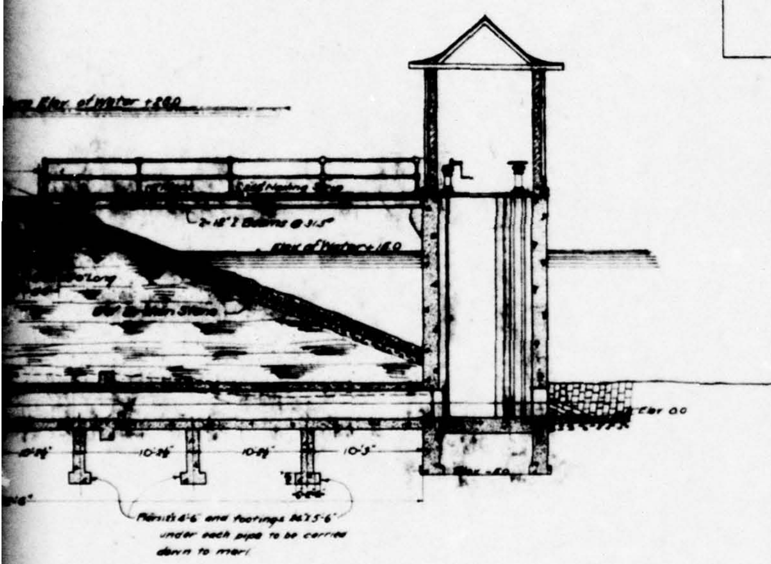


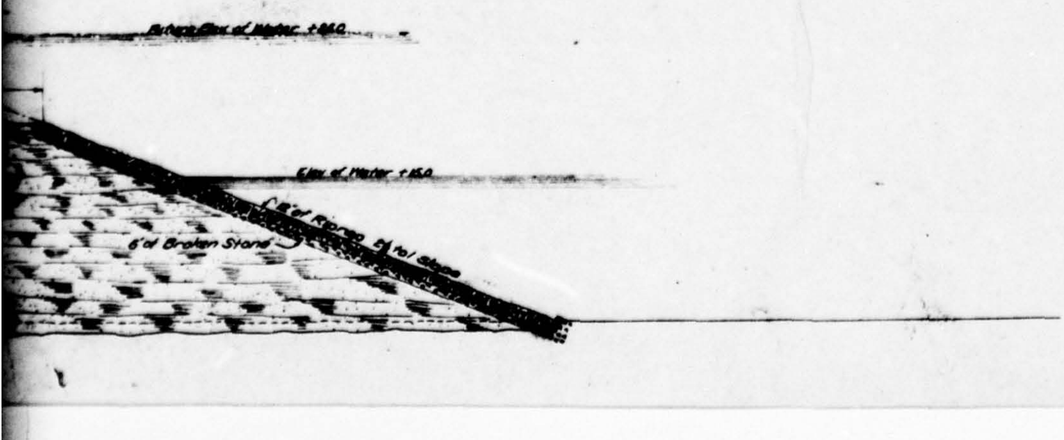
SECTION THRU 30" BLOW-OFF

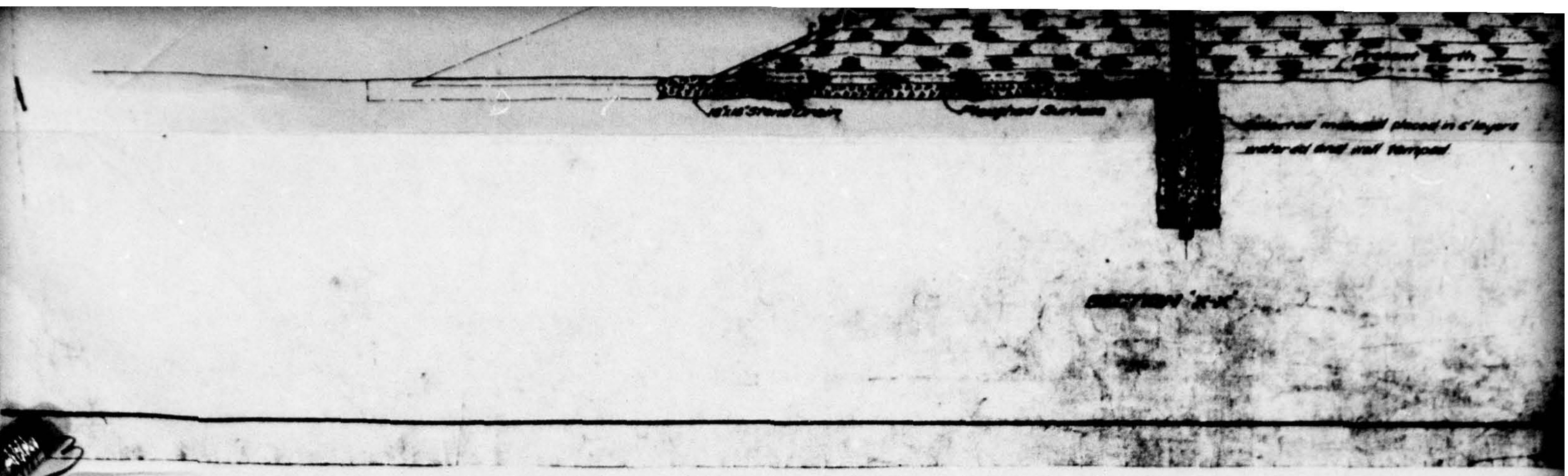




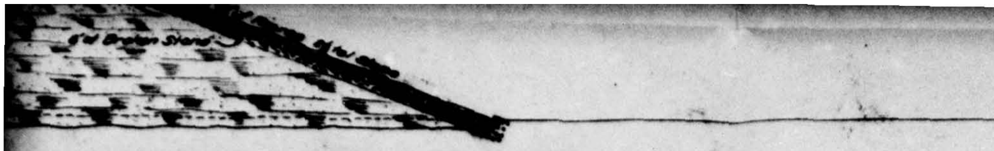
REINFORCED CONCRETE AROUND
30" DISCHARGE AND 30" BLOW-OFF







Stone Drain
Surface
SECTION 'X-X'



PORTSMOUTH, VA.
LAKE CAHOON DAM
CROSS SECTIONS

Approved by

Scale – [illegible] July 12, [illegible]
Revised [illegible]

Drawn H.R.B.

Traced H.R.B.

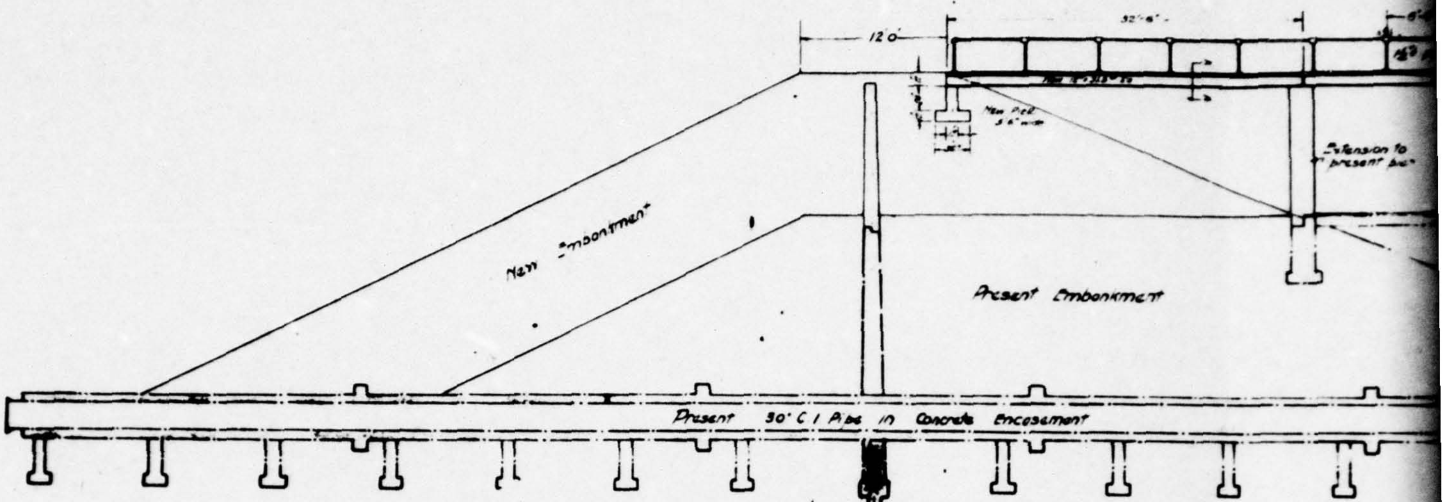
Checked W.L.K.

5-3-7

3[illegible]

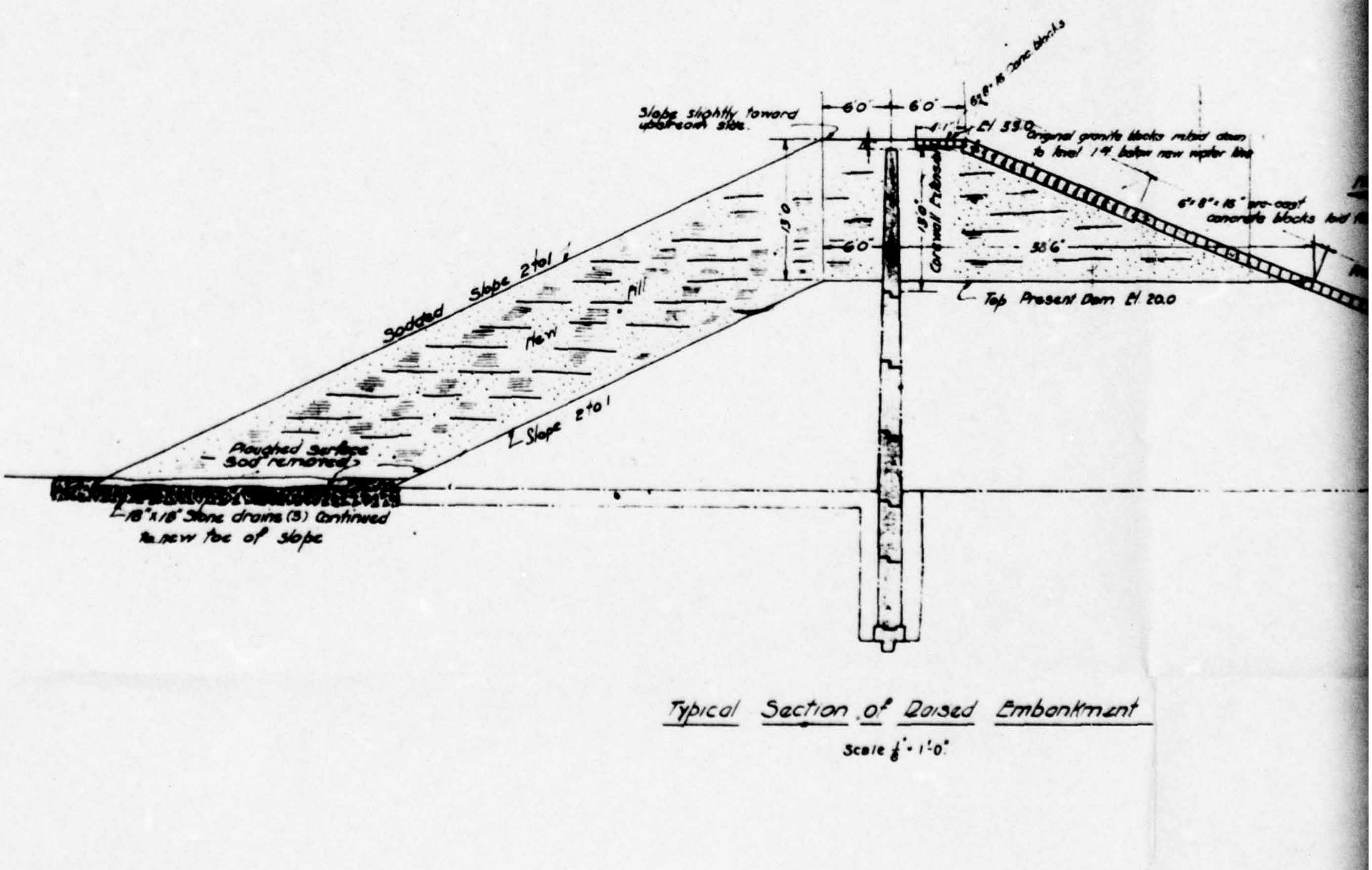
PLATE NO. 2

File 14-1



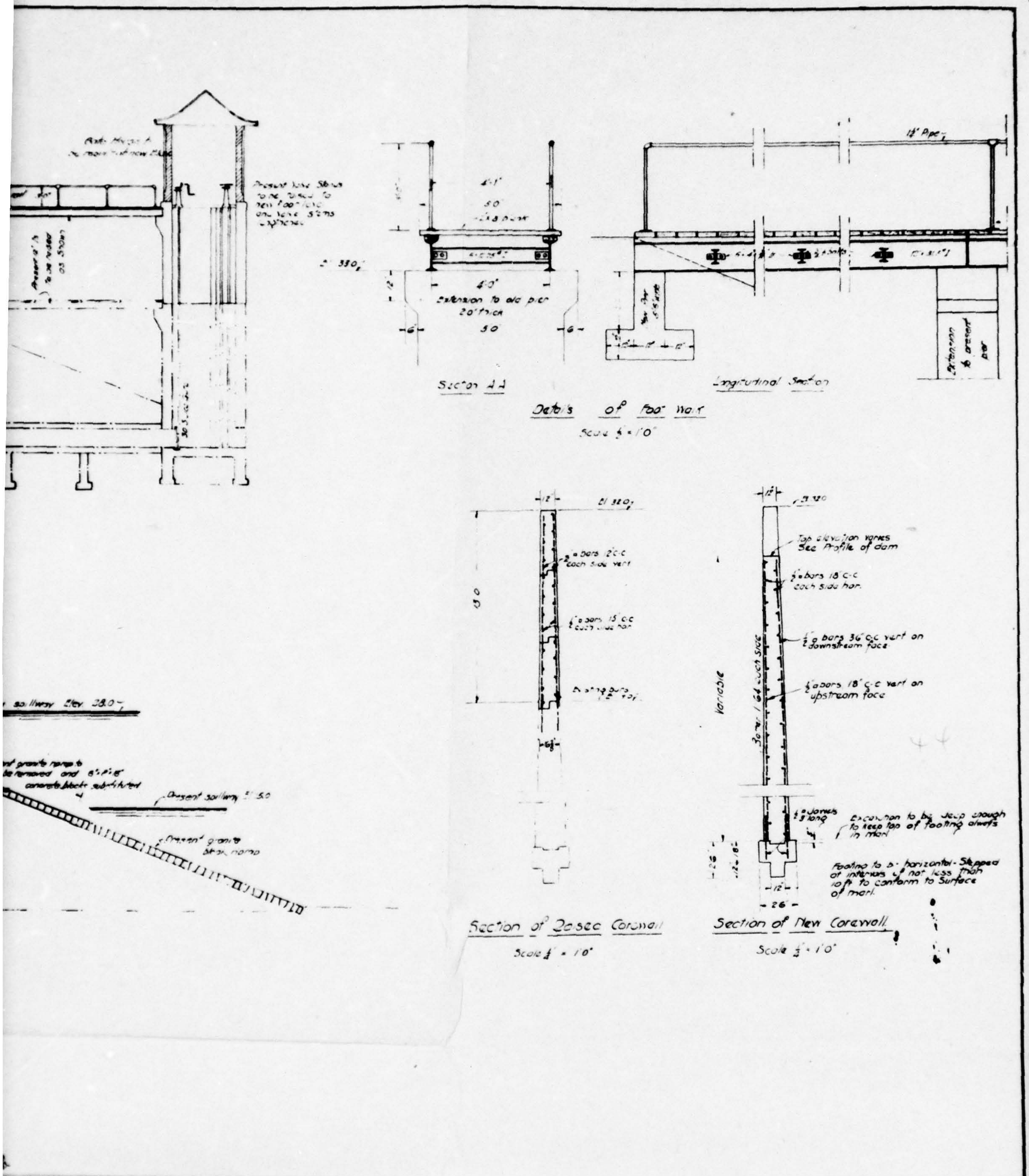
Section of Embankment Through Blow off Pipe

Scale 1/8" = 1'-0"



Typical Section of Raised Embankment

Scale 1/8" = 1'-0"


1½" Pipe
Extension to old pier 2'0" thick
Extension to present pier
Section A-A
Longitudinal Section
Details of Foot Walk
Scale ⅜" = 1'0"
Top elevation varies See Profile of dam
½" bars 36" c-c vert on downstream face
½" bars 18" c-c vert on upstream face
Variable
Excavation to be deep enough to keep top of footing always in marl
Footing to be horizontal - Stepped at intervals of not less than 10 ft to conform to Surface of marl.
Present spillway
Present granite block ramp
Section of Raised Corewall
Scale ¼" = 1'0"
Section of New Corewall
Scale ¼" = 1'0"

WAR DEPT.-CONSTRUCTION DIVISION.

PORTSMOUTH WATER DEVELOPMENT

— JOB-208 —

PORTSMOUTH, VA.

OFFICE OF CONSTRUCTING QUARTERMASTER

Details of Corewall & Embankment

SCALE AS INDICATED.

APPROVED BY [illegible]
Capt of Engineers Const. QM

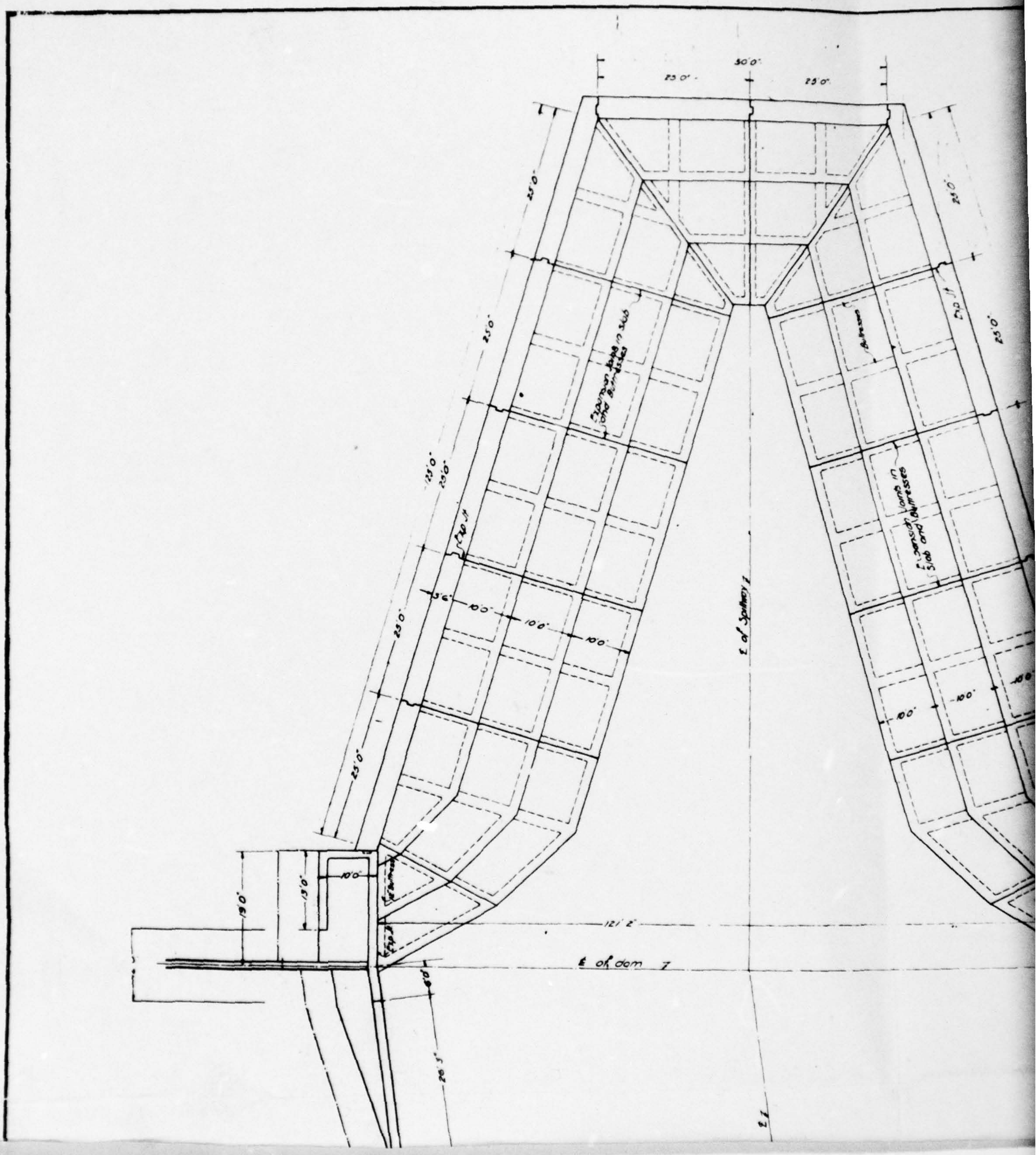
DATE 1-21-1919

REVISION APPROVED BY ____ DATE ____
____ DATE ____

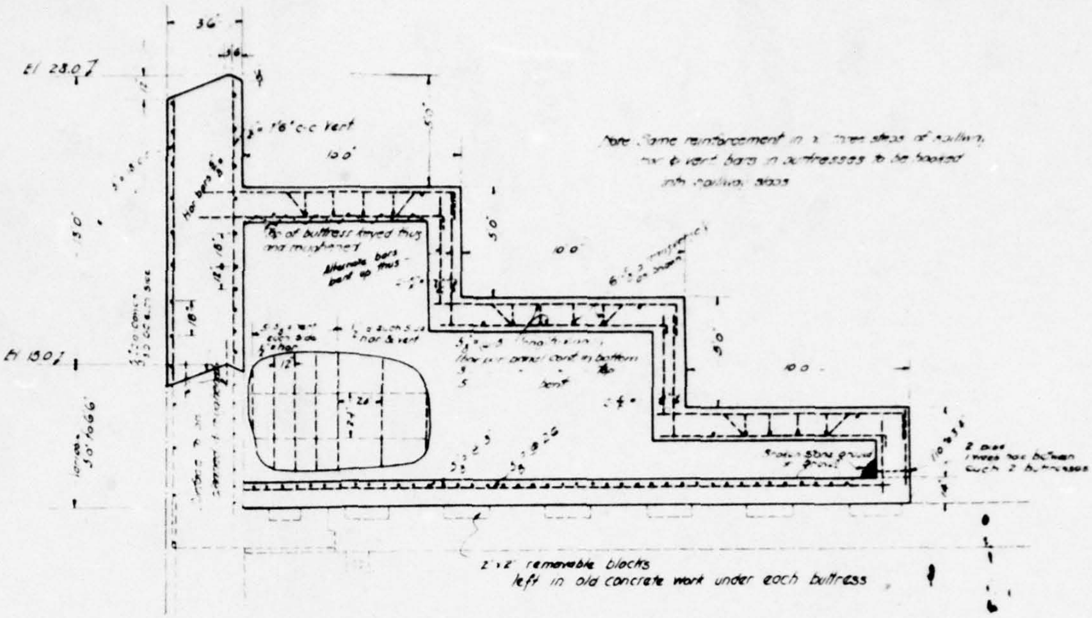
ULEN CONTRACTING CO. [illegible]

[illegible] R.C.G. [illegible] R.C.G. [illegible]

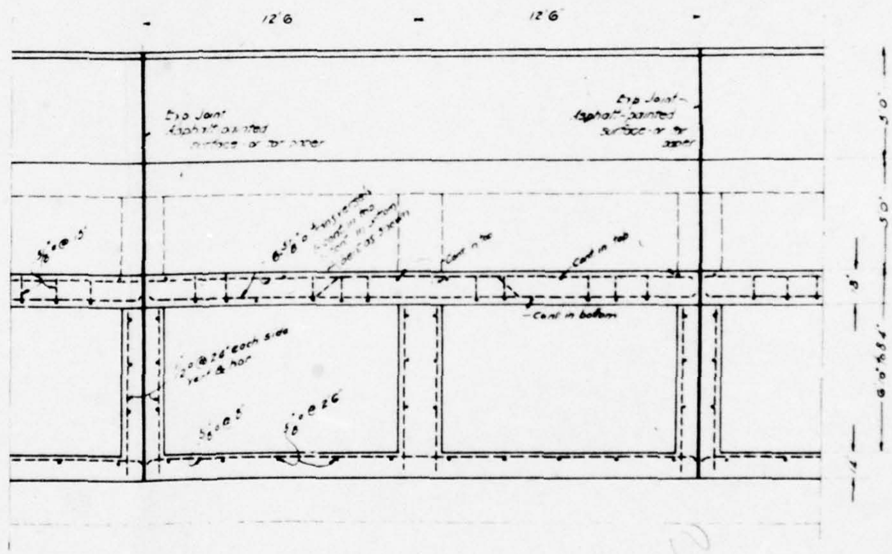
PLATE NO. 3



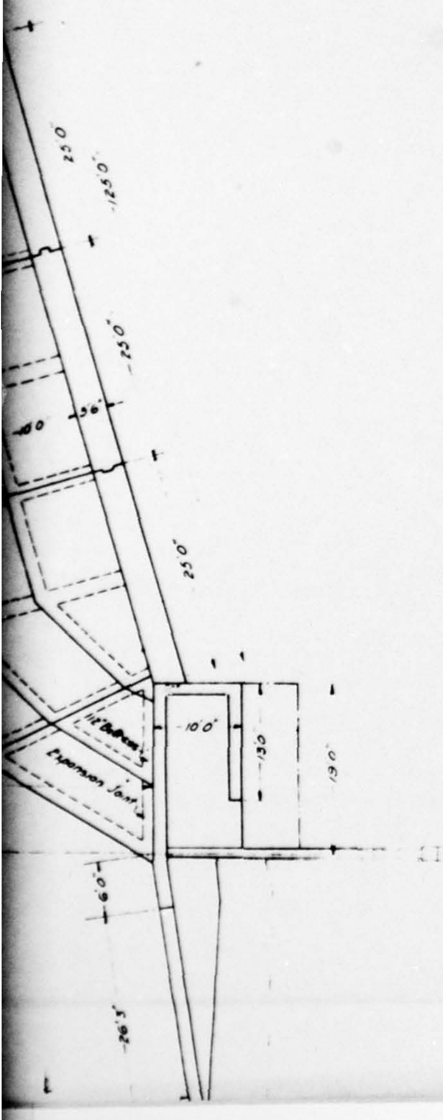
30'0"
25'0"
25'0"
Expansion Joints in slab and Buttresses
Buttresses
Expansion Joints in Slab and Buttresses
℄ of Spillway
10'0"
121' 2"
℄ of dam



Transverse Section

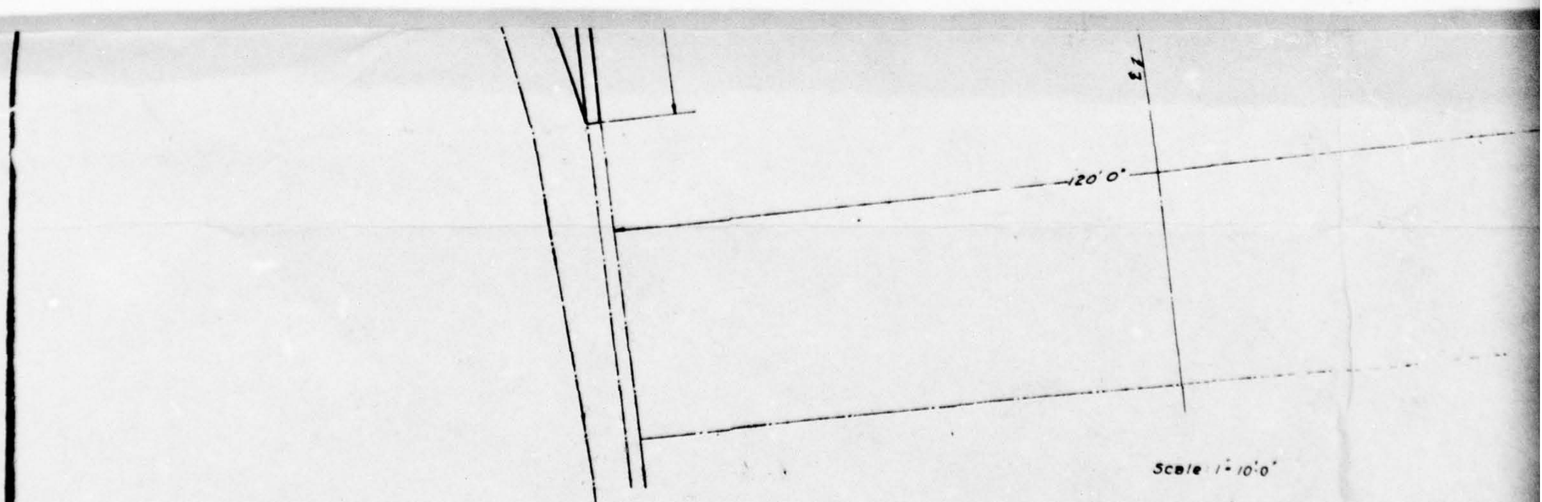


Longitudinal Section



Typical Details of Raised Spillway

Scale 1/4"=1'0"



3

REDUCED SCALE
HALF SIZE PLAN

WAR DEPT. - CONSTRUCTION DIVISION
PORTSMOUTH WATER DEVELOPMENT
— JOB-208 —
PORTSMOUTH, VA.
OFFICE OF CONSTRUCTING QUARTERMASTER

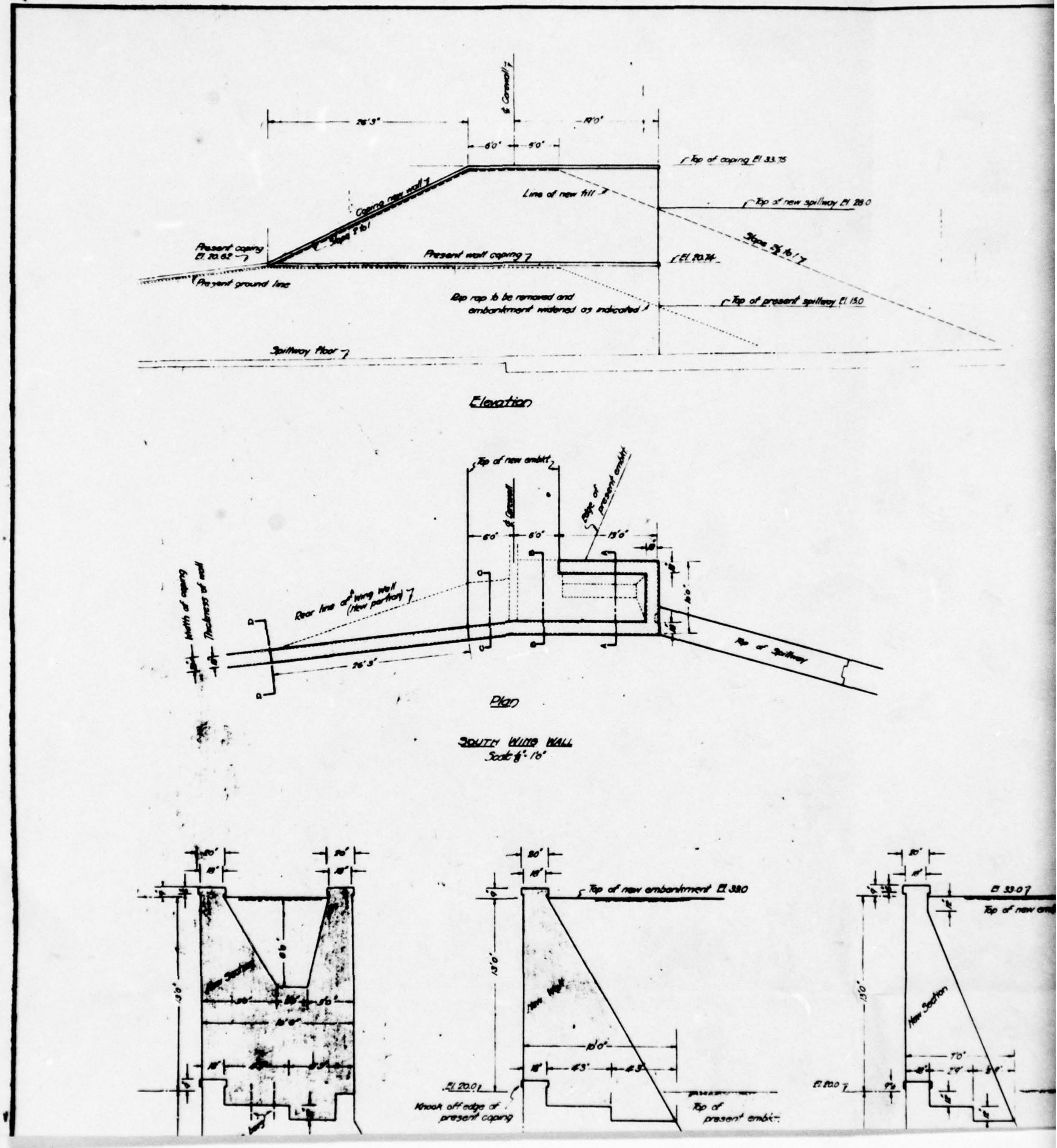
Details of Spillway
SCALE AS INDICATED

APPROVED BY
Capt. of Engineers Const. Q.M.
DATE 1-21-1942

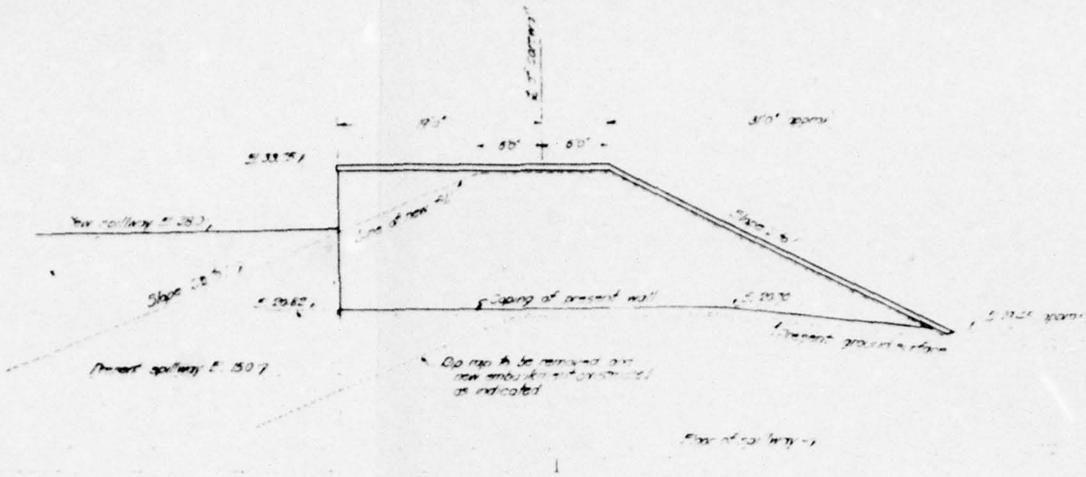
REVISION APPROVED BY
DATE
DATE

ULEN CONTRACTING CO.

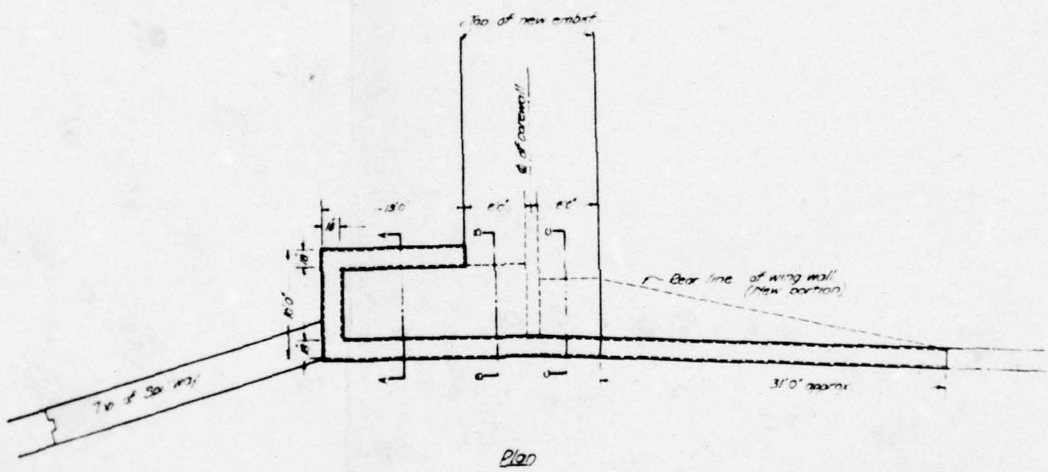
PLATE NO. 4



Top of coping El 33.75
Line of new fill
Top of new spillway El 28.0
Present coping El 20.62
Present wall coping
El 20.74
Present ground line
Rip rap to be removed and embankment widened as indicated
Top of present spillway El 13.0
Spillway floor
Elevation
Top of new embkt
Rear line of wing wall (new portion)
Top of Spillway
Plan
SOUTH WING WALL
Top of new embankment El 33.0
Knock off edge of present coping
Top of present embkt
New Section
New wall



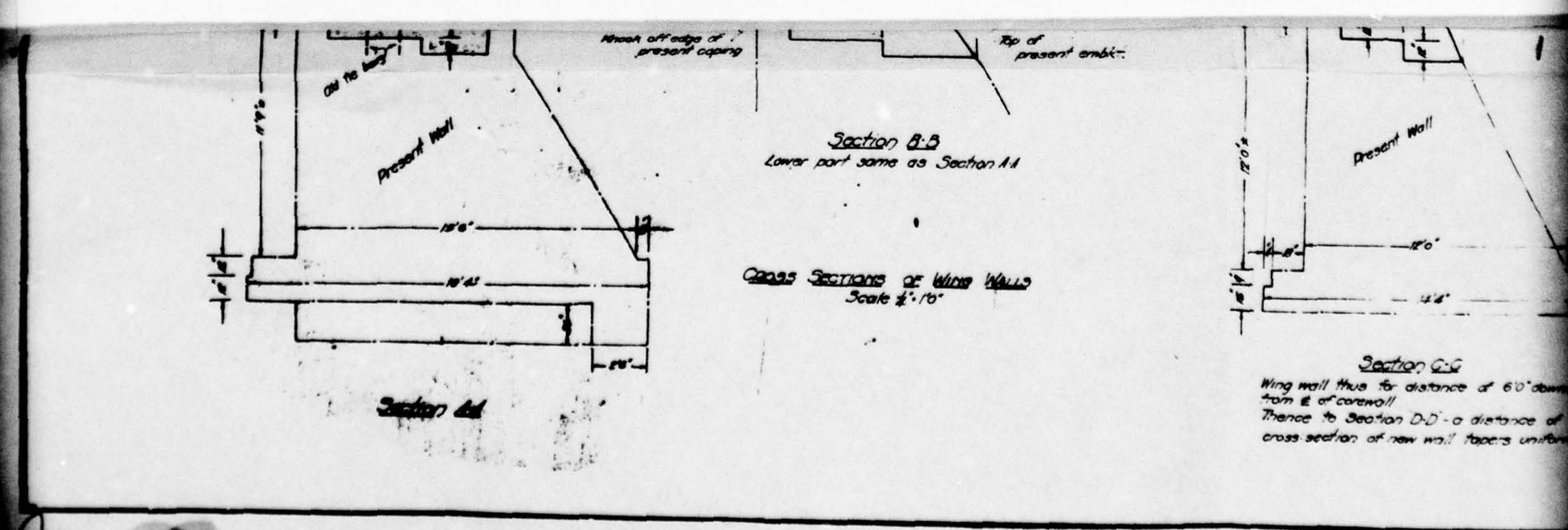
Elevation



Plan

NORTH WING WALL

Scale 1/8" = 1'0"


Knock off edge of present coping
Top of present embk.
Present Wall
Section B-B
Lower part same as Section A-A
Cross Sections of Wing Walls
Scale ¼"=1'0"
Section A-A
Present Wall
Section C-C
Wing wall thus for distance of 6'0" down
from ℄ of corewall
Thence to Section D-D - a distance of
cross-section of new wall tapers uniform

Section D-D

...f 6'0" downstream

...istance of 26'3".

...ers uniformly to 25'0

REDUCED SCALE

HALF-SIZE PLAN

WAR DEPT.-CONSTRUCTION DIVISION.

PORTSMOUTH WATER DEVELOPMENT

— JOB-208 —

PORTSMOUTH, VA.

OFFICE OF CONSTRUCTING QUARTERMASTER

Details of Wing Walls.

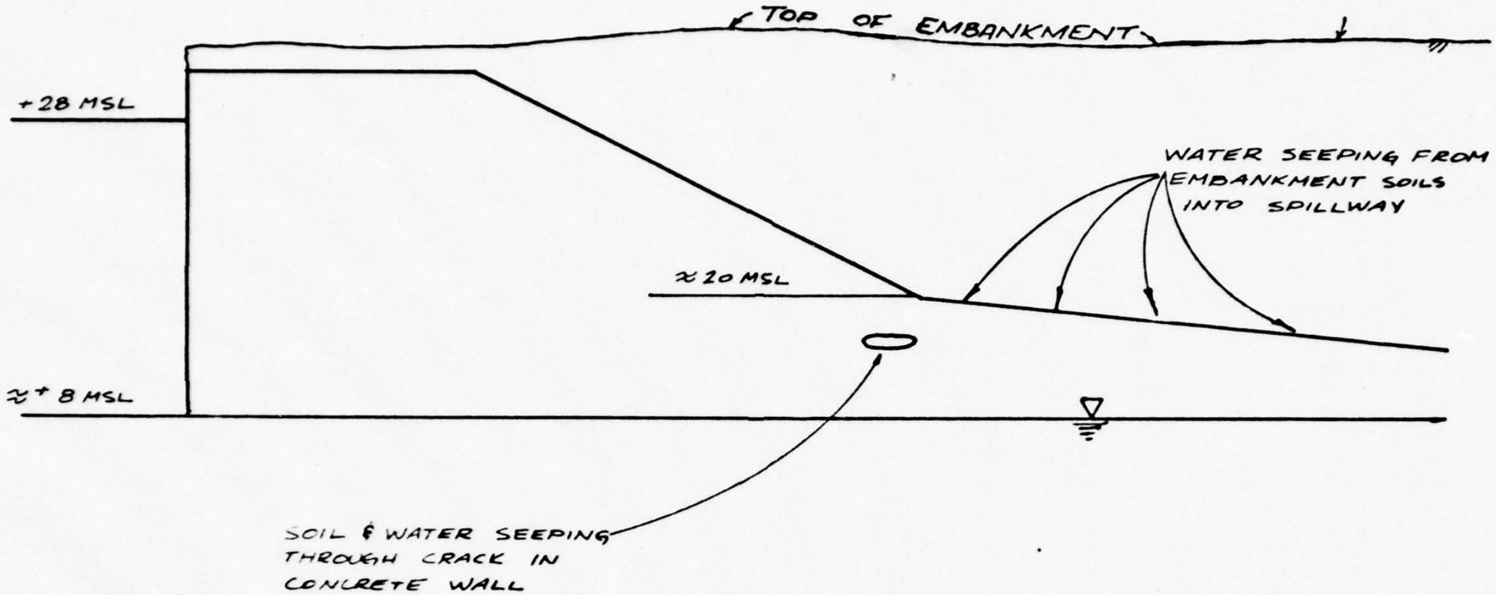
SCALE AS INDICATED

APPROVED BY [signature]
Capt. of Engineers Const. Q.M.
DATE 1-21-1919.

REVISION APPROVED BY
DATE
DATE

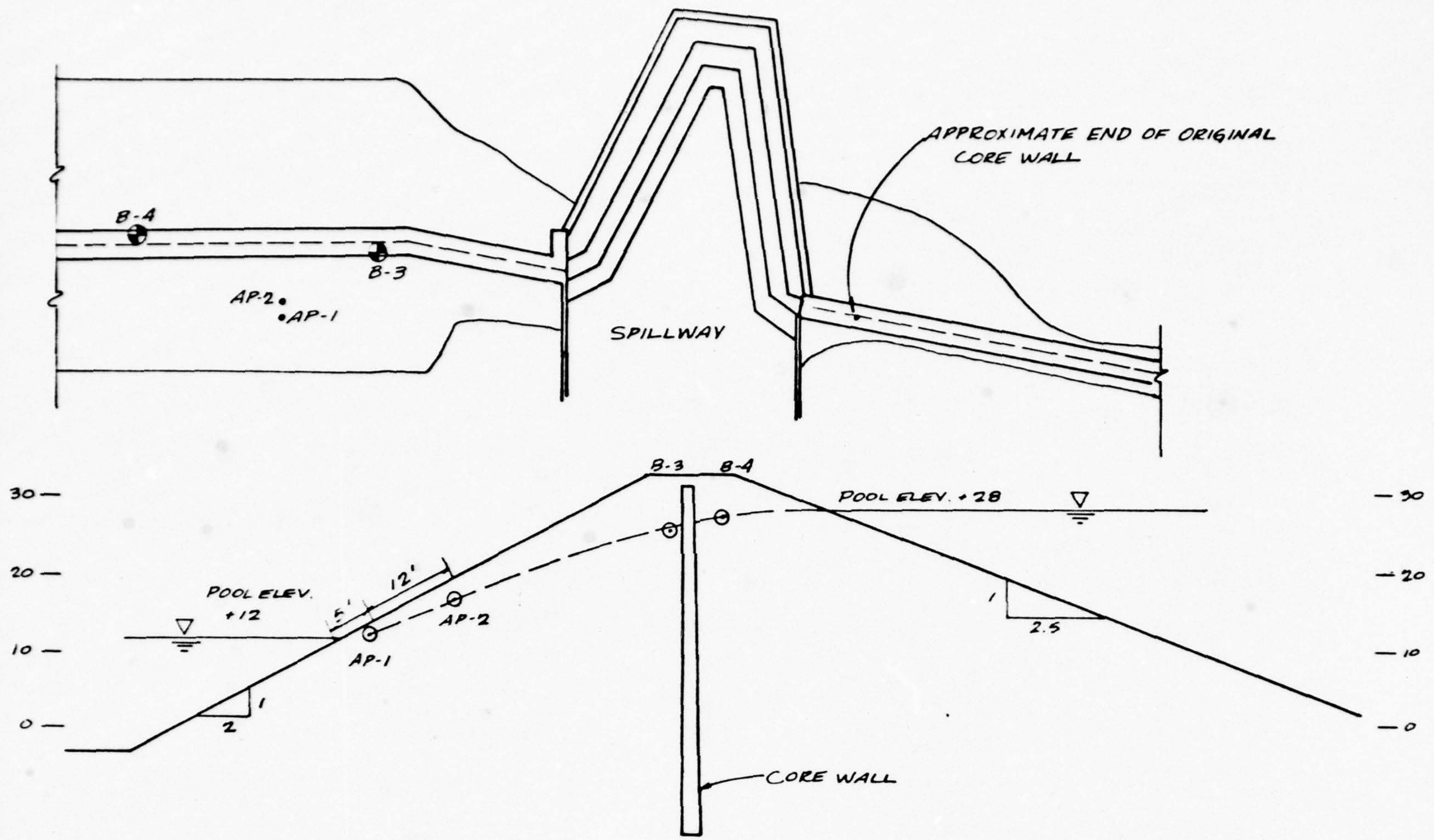
ULEN CONTRACTING CO. ENGINEERS & CONTRACTORS

PLATE NO. 5



SCHEMATIC OF NORTH SIDE OF SPILLWAY

OBSERVED SEEPAGE	LAW ENGINEERING TESTING COMPANY GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS 7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101		
LAKE COHOON DAM	SCALE	Drawn: DLB Checked: MJC	Job No W9-2270
NANSEMOND COUNTY, VIRGINIA	NONE	Date: 6-12-79	PLATE NO. 6



⊙ OBSERVATION POINTS WATER LEVELS

AP-1 2' } HAND
AP-2 3' } AUGER PROBES

B-3 7' } EXISTING OB-
B-4 53' } SERVATION WELLS

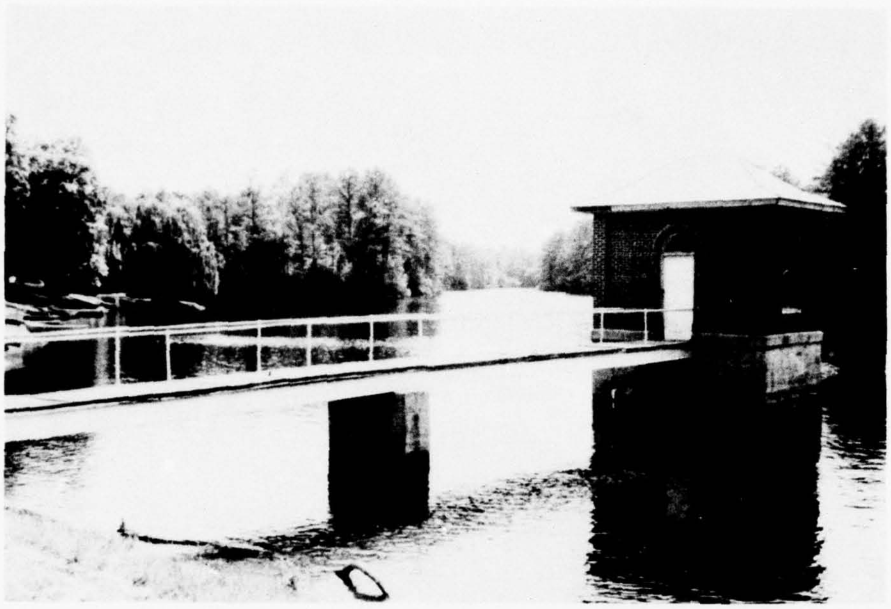
NOTE: WATER LEVELS TAKEN 5-7-79

WATER LEVEL OBSERVATIONS	LAW ENGINEERING TESTING COMPANY GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS 7913 WESTPARK DRIVE, MCLEAN, VIRGINIA 22101
LAKE COHOON DAM	SCALE NONE; Drawn: DLR; Checked: MJC; Date: 6-12-79; Job No. W9-2270
NANSEMOND COUNTY, VIRGINIA	PLATE NO. 7

APPENDIX II

PHOTOGRAPHS

LAKE COHOON DAM



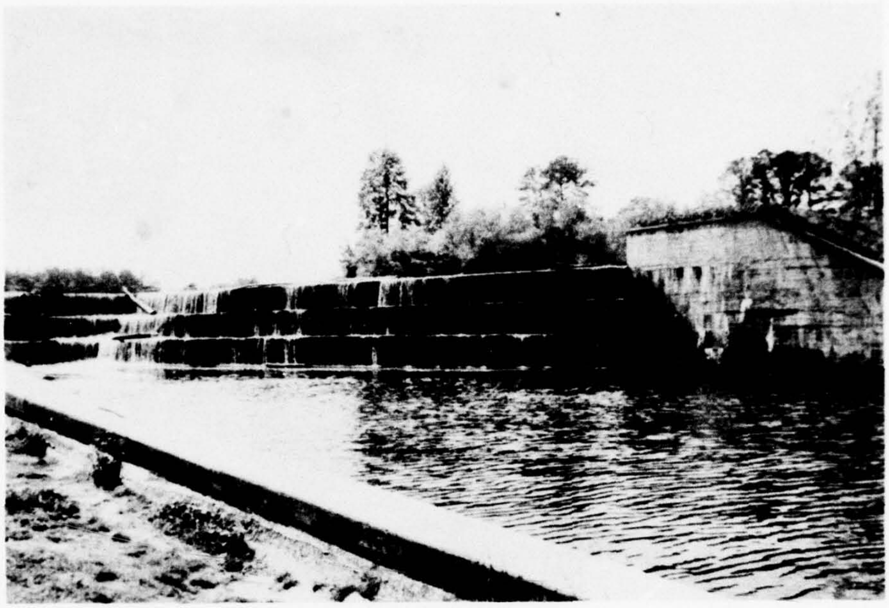
PHOTOGRAPH NO. 1
Intake Structure

60

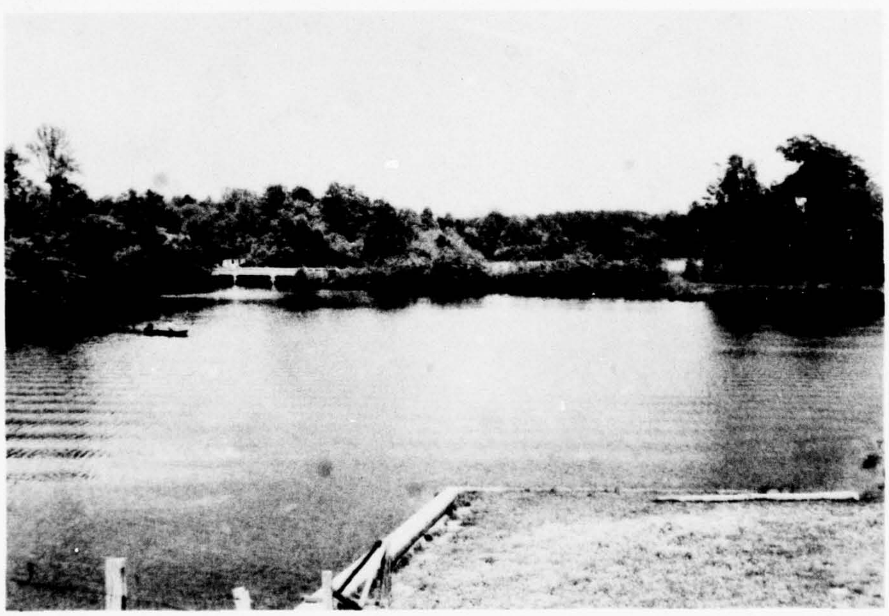
PHOTOGRAPH NO. 2
Upstream Face of Dam

PHOTOGRAPH NO. 3
Spillway

PHOTOGRAPH NO. 4
Face of Dam



PHOTOGRAPH NO. 5
Spillway Abutment



PHOTOGRAPH NO. 6
Downstream

APPENDIX III

FIELD OBSERVATIONS

Check List
Visual Inspection
Phase I

Name Lake Cohoon County Nansemond State Virginia Coordinates Lat. 3645.4 Long. 7637.8

Date(s) Inspection 5/7/79 Weather Clear Temperature 65^{o}+ F

Pool Elevation at Time of Inspection 28.2 M.S.L. Tailwater at Time of Inspection 12 M.S.L.

Inspection Personnel:

Leslie Nelms- Owner's representative Lake Cohoon Dam	Mike Cowell- Law Engineering, Soils & Geology	
Robert Gay, P.E.- SWCB		
Tan Young, P.E. Hydrology/ Hydraulics DMM&A		

Paul Seiler, P.E. Recorder

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEE PAGE ON LEAKAGE	Left side of spillway wall, water seeping over top of wall and through joint at this point. The joint shows brown mud accumulation on face and small roots.	Trees were cut down at left side just back of this wall. This loss of trees may contribute to the seepage.
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	No indication of cracks.	
DRAINS	None visible.	
WATER PASSAGES	30" C.I. pipe through dam.	
FOUNDATION	Not visible.	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None visible.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	No cracks visible.	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	None visible.	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	No obvious misalignments visible.	
RIPRAP FAILURES	None visible.	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONSTRUCTION MATERIAL	Earth embankment Concrete core wall	
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Concrete deteriorated	
ANY NOTICABLE SEEPATE	Wall of abutment at spillway. See concrete masonry Page 1 of 2	
STAFF GAGE AND RECORDER	None	
DRAINS	None visible	
FOUNDATION	Not visible	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Not visible.	
INTAKE STRUCTURE	Bridge to riser not advised to use at time of inspection; See Appendix IV Picture Pg. 26 & 28.	
OUTLET STRUCTURE	Not visible.	
OUTLET CHANNEL	Outlets into tailwater pool.	
EMERGENCY GATE	None.	

UNGATED SPILLWAY
(Emergency Spillway)

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Spalling of spillway steps and abutment walls	Needs referberishing to avoid more deterioration.
APPROACH CHANNEL	Water flows over 120-foot V-shaped spillway.	
DISCHARGE CHANNEL	Discharge into tailwater pool.	
BRIDGE AND PIERS	None.	

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None.	
OBSERVATION WELLS	Installed on top of dam 1978. Water standing.	
WIERS	None.	
PIEZOMETERS	None.	
OTHER		

70

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Flat, forested.	
SEDIMENTATION	Not visible.	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	No obstructions.	
SLOPES	Flat	
APPROXIMATE NO. OF HOMES AND POPULATION	Approximately 13 houses, estimated population of 30 people.	

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

ITEM	REMARKS
PLAN OF DAM	See Appendix I (plan)
REGIONAL VICINITY MAP	See Appendix I
CONSTRUCTION HISTORY	None available.
TYPICAL SECTIONS OF DAM	See Appendix I (plan)
HYDROLOGIC/HYDRAULIC DATA	See report.
OUTLETS - PLAN and - DETAILS	See Plans
- CONSTRAINTS and - DISCHARGE RATINGS	None on plans
RAINFALL/RESERVOIR RECORDS	None available.

ITEM	REMARKS
DESIGN REPORTS	
GEOLOGY REPORTS	See report prepared for owner 1978 by J. K. Timmons and Associates.
DESIGN COMPUTATIONS	None available
HYDROLOGY & HYDRAULICS	
DAM STABILITY	See Stability Section 6
SEEPAGE STUDIES	
MATERIALS INVESTIGATIONS	None available
BORING RECORDS	See plans and report
LABORATORY	See report
FIELD	
POST-CONSTRUCTION SURVEYS OF DAM	1978 by J. K. Timmons and Schnabel Engineering
BORROW SOURCES	Not known.

ITEM	REMARKS
MONITORING SYSTEMS	None.
MODIFICATIONS	See report.
HIGH POOL RECORDS	1.5' over crest by observer.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	See report.
PRIOR ACCIDENTS OF FAILURE OF DAM DESCRIPTION REPORTS	None on record.
MAINTENANCE OPERATION RECORDS	

ITEM	REMARKS
SPILLWAY PLAN	
SECTIONS	See plans
DETAILS	See plans
OPERATING EQUIPMENT PLANS & DETAILS	See plans

APPENDIX IV

PARTIAL REPORT, TIMMONS-SCHNABEL

1978

C. GEOTECHNICAL

1. Introduction

Our scope of services for the Lake Cahoon Dam included site inspection, review of existing design data, the drilling and logging of seven test borings, soil laboratory testing, and engineering analysis. The geotechnical engineering analysis included evaluation of site inspection, test borings, soil laboratory testing, geologic and related design data to develop the following:

a. Estimated subsoil profiles and groundwater levels for Cahoon Dam

b. Evaluation of Cahoon Dam embankment material properties.

c. Stability analysis of Cahoon Dam for maximum pool and other critical conditions developed during the study.

d. Report on findings concerning the condition of the Cahoon Dam with respect to geotechnical engineering conditions.

This scope of work corresponds to the U. S. Army Corps of Engineers Phase I and Phase II Studies outlined in "Recommended Guidelines for Safety Inspection of Dam" with respect to geotechnical engineering.

The general layout of the dam and spillway obtained from the design drawings is included in Drawing F1 included in Appendix F. The initial inspection phase included development of the regional geology for the site and review of the existing design drawings. This review was followed by a visual inspection of the dam.

2. Phase I Study

a. General Information

The Lake Cahoon Dam is an earthen embankment dam located on Cahoon Creek about 600 ft west of North Pitch Kettle Road in Suffolk, Virginia. The dam

was originally constructed in about 1912 to El 20 (El 0.00 P.W.D. = USC & GS MSL Datum El 1.93) and raised to El 33 in 1919 with addition of downstream berm. Normal pool level is El 28 in Lake Cahoon and El 12 in Lake Meade on the downstream slope. The downstream toe was flooded with construction of Lake Meade in 1959. A concrete cutoff wall exists beneath the center of the structure. The main portion of the dam is approximately 780 ft in length with maximum height of about 34 ft from the former bed of Cahoon Creek to the top of the dam. The maximum water depth at normal pool is approximately 29 ft. The principal spillway is located adjacent to the north abutment as shown on Drawing F1. Two low head embankments extend to the north of the north abutment with lengths of about 180 and 635 ft.

b. Regional Geology

A geological study was made of the immediate area in order to determine the type of soils which underly the dam and whether or not any faults are present. This study was performed by reviewing readily available geologic literature.

The dam is located in the Coastal Plain physiographic province and is underlain by the Yorktown Formation of Miocene geologic age. The Yorktown consists generally of preconsolidated marine sand, clay and broken shell material. West of the Nansemond River the upper 5 to 10 ft of the Yorktown Formation is usually orange to yellow in color before grading into its characteristic gray to green color. Surrounding hilltops in the immediate dam area are usually capped with the Sedley Formation of Pliocene geologic age. The Sedley is composed of fine sand and silty sand with thin layers of silty clay. This formation averages about 10 ft in thickness and was not encountered in the test borings.

The dam is located in an area where the probability of seismic activity is low and is expected to cause only minor damage. Specifically the dam is located in a Zone 1 seismic area as defined by the U. S. Army Corps of Engineers.

c. Review of Available Design Data

Three design drawings related to embankment and spillway construction were provided for review. These drawings titled "Core Wall", "Cross Sections" and "Section of Spillway" are dated June 26, 1912, July 12, 1912 and July 1, 1912, respectively. We understand the dam was constructed shortly after the dates of the drawings. We have prepared both longitudinal and transverse cross sections of the dam from these plans and these are designated Drawings F2, F3, and F4.

The dam consists of an earthen embankment and concrete cutoff wall. The plans indicate the soil was placed in 6 to 8 inch layers and rolled, although no density requirement is indicated. Side slopes are 2 horizontal to 1 vertical downstream and 2.5 horizontal to 1 vertical upstream. An initial crest width of 38'6" was planned at El 20. The normal pool level was planned at El 15 and the upstream slope was riprapped from the toe to the top of the dam. Riprap is indicated to consist of a 6 inch filter layer and 12 inch stone surface course. Three underdrains, 18 inches square in section, extend from the base of cutoff wall described below to the toe of the dam.

A 30-inch diameter discharge pipe and 30 inch diameter blow off pipe are indicated to run from the gate house to the down stream face under the dam along the south abutment. Both pipes are supported on spread footings spaced at about 10 ft on center and appear from the original drawings to be encased in concrete. Footings are indicated to be supported in the "marl" designated Stratum C in this report. These pipes also appear to be provided with concrete seepage collars spaced at about 30 ft on center. Both pipes were designed to extend about 40 ft downstream from the toe of the original dam to allow for raising the dam. The drawings indicate the pipes were laid below the original ground surface with invert at about El 0.

The concrete core wall is indicated to be about 1.5 ft thick at the top and about 2.0 ft thick at footing level. The top of the wall extended to El 19 with provision for additional extension of the wall at the time the dam was to be raised. The wall footing is indicated to extend to the "marl" soils of Stratum C. The wall originally extended to the horizontal limits shown on Drawing F1 and reinforcing steel was allowed to project for the planned later expansion of the dam both horizontally and vertically.

The spillway foundations are indicated to penetrate to the marl soils of Stratum C. The crest of the original spillway was at El 15. The 1912 plans indicate the spillway to be raised to El 28 at a later date. Detailed discussion of the spillway is contained in the Civil-Hydrology portion of this report.

Design drawings dated January 21, 1919 indicate the details required for raising the dam as contemplated on the 1912 drawings. The drawings used in this evaluation included Drawing A-1, "Plan and Profile of Dam", Drawing A-2, "Details of Core Wall and Embankment" and Drawing A-3, "Details of Wing Walls".

The embankment was raised to El33 by adding additional soils to the top and downstream slope. The original side slopes were maintained and the crest width was reduced to 12 ft. In the vicinity of the spillway, the original embankment narrowed to 15 ft in width as the spillway was approached on both sides. In these transition areas, the new embankment also extends beyond the original upstream slope as well as beyond the downstream slope. The three underdrains are also indicated to be extended to the downstream toe of the new dam. The lateral extent of the dam was also increased, especially to the north of the spillway as illustrated on Drawing F1.

The core wall is indicated to be extended laterally into the abutments and vertically to El 32. The core wall is indicated to extend to Station 6+00 south of the spillway and to Station 2+00 north. The spillway, riprap on the upstream slope and gate house were raised to accommodate the new pool elevation, El 28.

d. Field Inspection

(1). Settlement and Slope Stability

The embankment crest and side slopes were inspected in August 1978. No localized settlement, depressions or sinkholes were noted although some undulation of the downstream surface exists. These undulations are the result of differential settlement of the embankment soils because of variations in the compressibility of the foundation soils. The only major variation in the surface occurs along the downstream toe where a path has been developed by fisherman. No surface cracks were evident which

would indicate immediate stability problems. However, since a design stability analysis was not available for review, we recommended an evaluation of the dam's stability be included in this study. The results of the analysis are included in Section 3 of the report, the Phase II Study.

(2). Seepage

No seepage was observed on the downstream surface or toe of the dam either to the north or south of the spillway.

(3). Drainage System

Since this dam has a cutoff wall, seepage from the upstream Lake Cahoon to the downstream toe would be expected to be minimal. The original designer did, however, provide three underdrains from the cutoff wall to the toe and these were extended when the dam was raised. They have subsequently been submerged by Lake Meade and presently provide no useful function except in the case where Lake Meade is lowered below El 0.

(4). Slope Protection

The embankment was observed for wave and surface runoff erosion. No major erosion problems were detected. The embankment appears well maintained and continuing maintenance will prevent future erosion problems. Riprap on the upstream surface appears in excellent condition south of the spillway. However, south of the spillway, underbrush and up to 6-inch diameter trees have been allowed to grow resulting in displacement of some of the surface protection.

The short embankment section to the north of the spillway should be cleaned of underbrush and trees and the slope protection should be replaced.

3. Phase II Study

a. Subsoil Conditions

Test borings B-1 through B-4 were drilled along the crest of the dam and B-5 through B-7 along the downstream toe of the structure. Water observation wells were installed in test borings B-3 and B-4 to allow continued monitoring of the water level through the dam. Bulk samples and undisturbed soil samples were obtained from test borings B-1 through B-4 to evaluate material properties.

The test borings were drilled by Ayers and Ayers, Inc., Richmond, Virginia, and logged by our personnel. The test boring logs are included in Appendix 3 and data are projected on the dam sections included on Drawing F2, F3 and F4 included in Appendix F. Based on the test borings, the following generalized soil strata underlie the site to the depths indicated:

Stratum A: From the ground surface to depths of 14 to 39 ft	Brown to gray fine to coarse silty to clayey sand, EMBANKMENT FILL (SM), trace organic matter and gravel, with shell fragments; very loose to firm (N=2 to 28)
Stratum A_1: Interbedded with Stratum A to depths of 4 to 20 ft, maximum penetration	Brown to gray fine to medium silty clayey sand (SC) and silty clay (CL), EMBANKMENT FILL, trace organic matter, gravel and shell fragments; soft to very stiff consistency (N = to 25)
Stratum B: Below Stratum A to depths of 20 to 39.5 ft	Brown, gray to green fine to coarse SAND (SM), trace silt, organic matter and gravel, and fine clayey SAND (SC), trace gravel; loose to firm (N=4 to 16)
Stratum C: Below Stratum A and/or B to depths of 20.5 to 55.5 ft, maximum penetration	Gray to green fine to coarse clayey to silty clayey SAND (SC), fine sandy CLAYEY SILT (ML) and SILTY CLAY (CL), some fine sand with shell fragments; stiff to hard consistency (N = 7 to 57)

The depth of topsoil varied from 1 to 3 inches as indicated on the boring logs. N-values indicate the low and high Standard Penetration Test resistances encountered in a particular layer as determined from the number of blows required to drive a 2 inch O.D. 1-3/8" I.D. sampling spoon one foot using a 140 pound hammer falling 30 inches. This test is conducted after seating the sampler six inches in the bottom of the hole according to ASTM D-1586.

The soils of Stratum A and A_1 represent fill materials of the earthen embankment. Stratum B includes the recent geologic age deposits of Cahoon Creek. The soils of Stratum C are part of the Yorktown Formation of Miocene Age. The abutments of the dam probably consist of Sedley Formation soils. These soils were not encountered in the test borings.

b. Soil Laboratory Testing

Five undisturbed tube samples, five bulk samples and numerous jar samples were tested and data is presented in the Summary of Soil Laboratory Tests included in Appendix D. Soil classification is by the Unified System, ASTM D-2487.

(1). Stratum A - Fine to Medium and Fine to Coarse Sand (SM), Some Silt or Clayey Silt, Trace Fine Gravel, with Shell Fragments (Embankment)

The soils of this stratum are variable with fines content ranging from 15.7 to 37.5 percent. Natural dry densities ranged from 96 to 106 pcf. Natural moisture contents varied from 11.5 to 26.2 percent. A permability test indicated a value, $k=2.4 \times 10^{-6}$ ft/min or very low.

A consolidated undrained triaxial compression test was performed to evaluate the soils total shear strength parameters. The following values were obtained.

Angle of Internal Friction, $\phi = 15^o$
Apparent Cohesion, c= 400 psf

A slow direct shear test was also performed, to evaluate the effective strength parameters. The following values were used in our analysis:

Effective Angle of Internal Friction, $\phi' = 37^o$
Effective Apparent Cohesion, c'= 0 psf

Two compaction tests were performed according to ASTM D-698 to develop representative maximum dry densities for the embankment soils. The maximum dry densities were found to be 118.6 and 119.7 pcf. The insitu densities are thus indicated to be only 80 to 89 percent of the maximum dry densities. These values are lower than what is normally required; however, it should be understood that tube samples in sands are subject to disturbance and densities obtained by this method are usually lower than actual insitu densities.

(2). Stratum A_1 - Fine Silty Clayey Sand (SC) (Embankment)

One sample of this stratum indicates the fines content to be 39 percent. The natural dry density was found to be 113 pcf and natural moisture contents varied from 15.2 to 16.7 percent. A permability test indicated a value $K=1.2 \times 10^{-6}$ ft/min or very low.

Total and effective strength parameters were not determined by laboratory tests. However, we have conservatively estimated these parameters based on data for soils of similar classification and density at Kilby Dam and for other projects in the general geographic area as follows:

Angle of Internal Friction, $\phi = 4^{o}$
Apparent Cohesion, c= 700 psf

Effective Angle of Internal Friction, $\phi' = 37^{o}$
Effective Apparent Cohesion, c' = 0 psf

(3). Stratum B - Fine to Medium Sand (SM), Trace Silt

Laboratory test were not performed on this stratum. However, strength parameters were estimated from the test boring Standard Penetration Test data, as follows:

Angle of Internal Friction, $\phi = 35^{o}$
Apparent Cohesion, c= 0 psf

Effective Angle of Internal Friction, $\phi' = 35^{o}$
Effective Apparent Cohesion, c' = 0^{o}

An estimated total unit weight of 115 pcf was also used in our analysis.

(4). Stratum C - Silty Clay, Some Fine Sand (CL) and Fine Sandy Clayey Silt (ML)

The fines content of this soil varies from 60.2 to 69.2 percent. Natural moisture contents were found to be ranged from 28.6 to 32.4.

The permeability of this soil is estimated to be less than the fill soils of Strata A and A_1. Based on laboratory test data in our files for soils of similar density, a very low value of $k=10^{-6}$ ft/min is probably typical.

c. Geotechnical Engineering Analysis

The basic design requirements included in the contract plans which were evaluated in more detail in this phase of the study included: (1) the foundation condition, (2) the effectiveness of the cutoff wall, (3) the material type and compaction, and (4) the stability of the dam.

(1) Foundation Conditions

The test boring data indicates the dam is founded on a thin layer of alluvial sands underlain by the Yorktown Formation soils of Miocene geological age. This foundation is relatively incompressible since the Yorktown Formation soils have been preconsolidated by sediment load which was subsequently removed by erosion during past geologic history. This formation provides an excellent base for support of the dam.

(2) Cutoff Wall Effectiveness

The permeability of the embankment soils required the dam designers to provide the structure with a concrete core wall to reduce flow of water through the dam and lower the phreatic level within the downstream portion of the dam. This wall is indicated on the original plans to be founded in the "marl" or Yorktown formation soils of Stratum C. In order to check the effectiveness of the core wall as a cutoff, permanent water observation wells were installed in borings B-3 and B-4. These wells indicate the water level downstream and upstream correspond very closely with the water levels of the pools downstream and upstream respectively. Thus, the core wall is performing very satisfactorily.

(3) Material Type and Compaction

The embankment soils are basically sand with variable amounts of clay and silt and have been separated in our analysis into Strata A and A_1. These soils have relatively high compacted strengths as indicated by the data provided in the previous section. The original drawings indicate the embankment was constructed in 6 to 8 inch lifts. Each lift was required to be watered and rolled or compacted. This was common practice at the time the dam was constructed. Watering was believed to help compact

a sandy soil; however, the rolling was probably more effective. Laboratory standard density tests and field density tests for control of compaction were not developed at the time the dam was constructed.

Undisturbed samples of the embankment soils indicate material is poorly compacted. However, tube samples are subject to disturbance in sands and we believe the embankment soils have a higher degree of compaction than indicated by these densities. The strength test data and standard penetration test data verify the soils have been compacted because of the relatively high angles of internal friction.

(4) Embankment Stability

A stability analysis was performed on both the upstream face and the downstream face. Since both faces are partially submerged under normal operating conditions, the sudden drawdown from spillway crest, designated Case I and the steady seepage condition, designated Case III, were evaluated. Embankment section CC illustrated on Drawing F2 and containing borings B-1, B-4, and B-6 was selected for analysis since this section is typical. The following factors of safety were obtained.

Surface	Case	Loading Condition	Factor of Safety	Required Min. Factor of Safety
Upstream	I	Sudden drawdown of Reservoir	1.3	1.2
	III	Steady Seepage	1.9	1.5
Downstream	I	Sudden drawdown of Reservoir	1.2	1.2
	III	Steady Seepage	1.5	1.5

These factors of safety were acceptable in accordance with the U. S. Army Corps of Engineers guidelines. Case II and IV described as a "partial pool case" and the "earthquake case" respectively were not evaluated as these are less critical. The critical circles associated with the above factors of safety are indicated on Drawing F2.

Under the influence of the maximum probable flood, as described previously in the Civil-Hydrology section of this report, water in the Cahoon reservoir will rise over the present top of the dam. The stability of the upstream slope under this condition is less of a problem since the slope is completely submerged. Therefore, this case was not evaluated further. The

factor of safety for the downstream slope, under this reservoir condition will not be adversely effected since the core wall will maintain the water level downstream at the pool level for Lake Meade, with the planned addition of a concrete wall on top of the dam. Sliding stability was also considered. The factor of safety against sliding was found to exceed 4.

4. Conclusions and Recommendations

Based on the geotechnical engineering data contained in this report, the following summary of conclusions and recommendations is presented:

a. The Cahoon Dam contract drawings were reviewed for conformance with generally accepted principles of geotechnical engineering. We believe the design is suitable for the site foundation conditions and materials utilized.

b. The dam was inspected using the U.S. Army Corps of Engineers guidelines for settlement and slope stability, seepage, drainage systems and slope erosion problems. Although no major problems were observed, the short section of the embankment north of the spillway has not been maintained. This area contains trees and underbrush which should be removed. The slope protection of the upstream face should also be restored.

c. Since the dam was constructed with a very steep downstream face and was subsequently partially flooded with construction of Lake Meade, we recommended a Phase II study to evaluate the strength of the embankment soils and performance of the core wall as a cutoff to water flow through the dam. This study included drilling seven test borings, installing two permanent water observation wells to allow present and future monitoring of the water level through the dam, laboratory testing and engineering ananysis.

d. The water observation wells indicate the water level upstream of the core wall is at about El 24 and downstream is at about El 11. These data indicate the core wall is performing as planned. The water observation well located in Boring B-3 should, however, be monitored quarterly to determine any major variation from the downstream water level El 12. A rise in this level without a concurrent rise in the level of Lake Meade, such as under surcharge conditions, would indicate the core wall cutoff is not functioning properly. If this should ever occur, a rise in the water level on the downstream side could trigger failure of the downstream slope and the dam. Thus the water observation well should be monitored quarterly.

The water level should not exceed the level of Lake Meade by more than 1 ft. We should be notified if the water level rises above this level so that we may evaluate the seriousness of the problem.

e. A stability analysis was performed using shear strength data developed in the soils laboratory and dam geometry to determine the factors of safety of the embankment with respect to shear failure.

Surface	Case	Loading Condition	Factor of Safety	Required Min. Factor of Safety
Upstream	I	Sudden drawdown of reservoir	1.3	1.2
	III	Steady Seepage	1.9	1.5
Downstream	I	Sudden drawdown of reservoir	1.2	1.2
	III	Steady seepage	1.5	1.5

These factors of safety are acceptable and indicate the embankment is stable under present loading conditions.

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III. LAKE CAHOON DAM SAFETY INSPECTION

A. CIVIL - HYDROLOGY

1. Description of Dam

The original dam was constructed in 1912 . The original dam was an earth embankment with a gate house and flood gate which acted as the principle spillway. The earth embankment had a concrete core wall in the center. Top of original embankment was elevation 20.00 M.S.L. In 1919 the earth embankment was raised to elevation 33.00 along with the core wall. At that time a 300' long concrete spillway was added at the north end of embankment. The new spillway elevation is 28.00 M.S.L. The original gate house was raised to new elevation also. The embankment has grass covered sideslopes of 2:1. The stone underdrain system on the downstream side of embankment was continued under raised embankment. Embankment has two 30" cast iron pipes through it which were laid in existing channel location.

The primary purpose of this Lake and dam is to act as a water supply storage and allow the level of Lake Meade (downstream) to be maintained at a height to supply the water treatment plant located at Lake Kilby Dam. The capacity of the Lake is 1963 M.G. with 502 acres of flooded land.

- Dam Classification -

Dams are classified by the Corps of Engineers in accordance to size and hazard potential. The classification will be used to determine the recommended design flood that the spillway must pass. Lake Cahoon size classification is"Intermediate" since the storage capacity is greater than 1000 acre-feet and less than 50,000 acre-feet. The hazard potential classification is"Significant" since there will only be damage to isloated homes, secondary highways and minor railroads and service interupted to utilities.

By the Corps of Engineers' criterion, a Significant Hazard, Intermediate-Size dam must be able to pass the ½ probable maximum flood (PMF) through the spillway.

A probable maximum flood by Corps of Engineers' standards is a flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. Such a condition would be similar to that which occurred in Nelson County during Hurricane Camille. See Storm Intensity Comparison Chart following Physical Inspection section.

The State of Virginia adopted regulation No. 9 for impounding structures in March 1978 which was filed in May 1978 and became effective July 1, 1978. This regulation states that within one year from effective date, every owner of an existing impounding structure, shall provide data and information to the State Water Control Board sufficient to enable the Board to determine whether to issue an Operations and Maintenance permit for existing impounding structure, or to direct such work as may be necessary to mitigate extant hazard to life and property attributable to the existing structure. The procedures for this can be found in the copy titled, "Impounding Structure Regulations", Chapter 12, Page RB-6-11. (See Appendix). The sections which have been underlined in this regulation should be given attention since they pertain to requirements, findings, actions by the Board, right to hearing on suggestions, and enforcement.

As stated in the Regulation, if a formal complaint is filed due to unsafe conditions or operating conditions, and is found to be true by the Board, the owner shall be required to place the facility in a safe condition as suggested or the Board shall cause such action to be taken as breaching or removal of any impounding structure found beyond repair. As the guidelines indicate in this regulation, the following findings were acquired for this impounding structure titled, Lake Cahoon Dam.

2. Physical Inspection

a. Procedure

During the months of September and October, 1978, several field inspections were conducted by engineering personnel from J. K. Timmons and Associates. The water surface elevation was observed at two different levels during these inspections. At both levels the water was not flowing over the spillway crest line. A gate valve was open in the Gate House allowing the level to be kept down.

At the time the photographs were taken, the water level was approximately 15" to 18" below crest line of spillway. As can be seen in photographs, this showed leaks and wet spots on downstream side of spillway. The concrete was sounded on spillway and the cracks, leaks, wet spots and holes were mapped. The joints were probed and measured for depth, along with the holes.

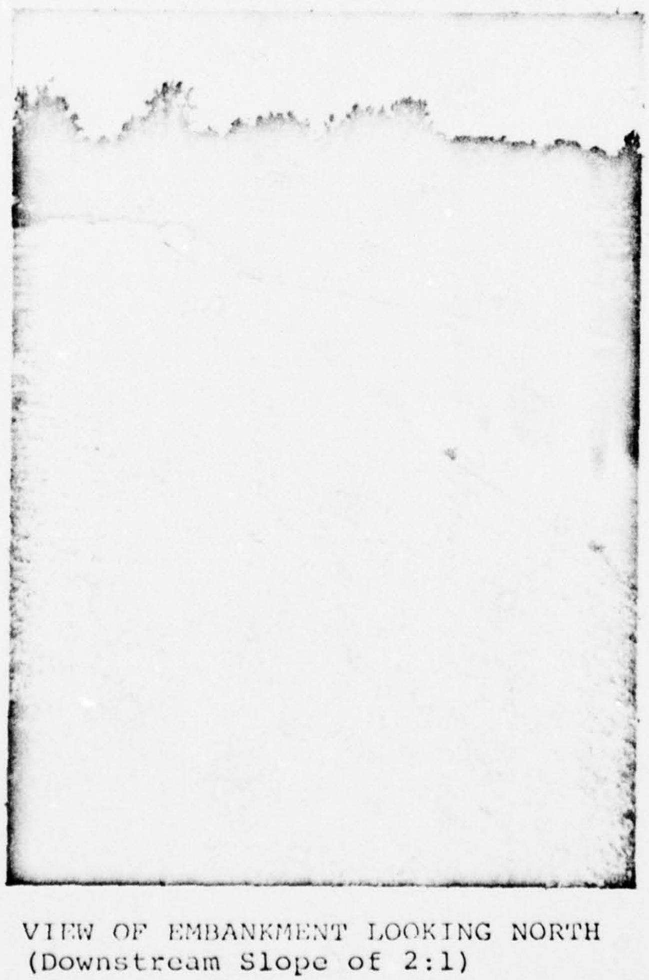
The embankment was walked to evaluate for erosion, sunken areas, and seepage. The Gate House was inspected for operational value and safety features. All of these operations and findings were photographed and are included in this report.

In general, an overall visual inspection was made of embankment, Gate House and concrete spillway.

(b) Findings

1. Dam Embankment

The embankment's visual condition appears to be good,as can be seen in the following pictures. The slopes on both sides are covered with a good stand of grass. There is a portion on the upstream side which is lined with stone;and although no severe sink holes can be seen, there are sections of the stone lining which have been moved and need to be relaid. From what can be seen, the 2:1 slopes show that no sloughing has occurred. A more definitive analysis of the embankment was made by the soil consultant (Schnabel Engineering). The Geotechnical section of this report contains data used for further analysis of embankment.



VIEW OF EMBANKMENT LOOKING NORTH
(Downstream Slope of 2:1)

2. Spillway

a. Concrete Spillway (January 1919 Plans)

The general condition of concrete is fair. The surface shows leaks, pot holes and concrete deterioration. This spillway is a bulkhead type of construction which can be seen in transeverse section on Cracks Location Map in Appendix of this report. Appendix should be consulted along with photographs which show the location and visual condition or the findings. The same code as shown on Page I-A-3 was used in describing the cracks.

P.H. - Pothole - Requires Repair
D.C. - Dry Crack - Requires Sealing
W.C. - Wet Crack - Requires Cutting Out and Sealing
L.J. - Leaky Joint - Requires Sealing
B.J. - Bad Joint Compound - Requires New Joint Compound
H - Hole Over 2" Deep - Requires new concrete and Tie to Old Surface
B.C. S. - Bad or Flaking Concrete Surface - Requires that surface should be Scaled and Plastered.

The retaining walls of embankment show a rotten surface for the concrete. See Photographs 22 and 39 (Northern Wall) and Photographs 40 and 41 (Southern Wall). As indicated in Appendix II, the southern section of spillway shows several leaks and wet spots. On this same face, photographs 35 and 38 show that the reinforcing bars, originally imbedded 6" into the surface of bulkhead, are not only exposed, but hanging in mid-air in places. The wall shows leaks in several places and a water spout in one spot extending from the surface of wall. (Photographs 30, 31, 33, and 34) The end or western section of spillway at the time of inspection was wet and covered with slime. The joint in the middle indicated a bad leak at the time of photographing. However, on another inspection in October, when the water was down approximately 5' to 6', this joint was sounded with

a hammer and the rotten surface removed, exposing what appeared to be soil which had lodged in this joint to help stop the leak.

The northern spillway section showed much the same as the southern section, although not as much leakage was present. The Cracks Location Map should be consulted in Appendix for a true overall picture of the problem areas. The southern retaining wall has a small fence atop it, but none could be seen on the northern wall.

b. Gate House

The Gate House which was raised in 1919, appeared to have a sound structural base and wet wells. The brick and mortar structure seems to be in good condition. The exterior roof is slate and shows some sections which are broken. The exterior wooden trim requires painting and replacement of rotten sections. The building has three windows, all of which need new panes and wooden sections requiring repair. They have also been dislodged from their masonry openings. The door has not been painted in some time and has signs of weathering.

The ridge roll is no longer atop the structure as shown on original drawings. Appendix (Gate House Drawing and Pictures) shows these problems. No exterior lighting is present. The wooden and steel bridge to Gate House has rotten planks and also shows that the steel could use a coat of paint. The handrail is wobbly and unstable. At the entrance to bridge is a farm wire type gate which serves little purpose.

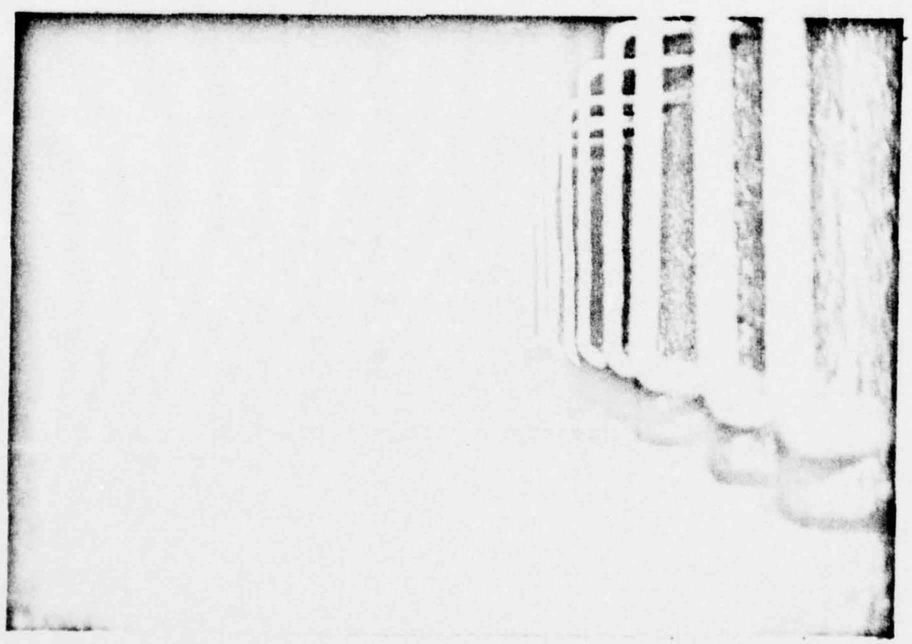
The interior of the Gate House shows a heavy steel grate floor which is rusted. The interior wooden trim shows much the same as the exterior, in that it requires repair and painting. Although some electrical wiring is present, the power has been cut outside the building and the interior wiring is not in usable condition. The two 30" sluice gates below water surface were not visible in the wet well due to still water atop them. However, the 24" gate, approximately 11 ft. below grating, appears to be in good condition. It was observed that the gate stand to the right of door was cracked to a great degree. The gate to the left of door was tried and found to have a gear ratio of almost 1 to 1, which makes it difficult to open. The gate valves to the

west or back side of building seem to be in poor condition also. The 24" valve was open allowing the water level to be lowered. In an attempt to see the condition of the lower valves, we tried to close the 24" valve which was open, but after some 100 or so turns, it was discovered that the valve had some wood lodged in it and could not be closed. The floor grating was removed and personnel tried to descend the ladder in the wet well to dislodge the wood. The ladder was found to be in an unsafe condition. The rungs which are steel rods have rusted to the point that they will not support a man's weight. Also, several items were piled on grating which would not allow all grating to be removed. The trash rack on the exterior of building was observed from the interior of the building by opening the window and was found to be in a worn condition. The appendix of this report shows drawing and photographs of this building and spillway which may be consulted for a pictoral view of these conditions along with the following photographs.

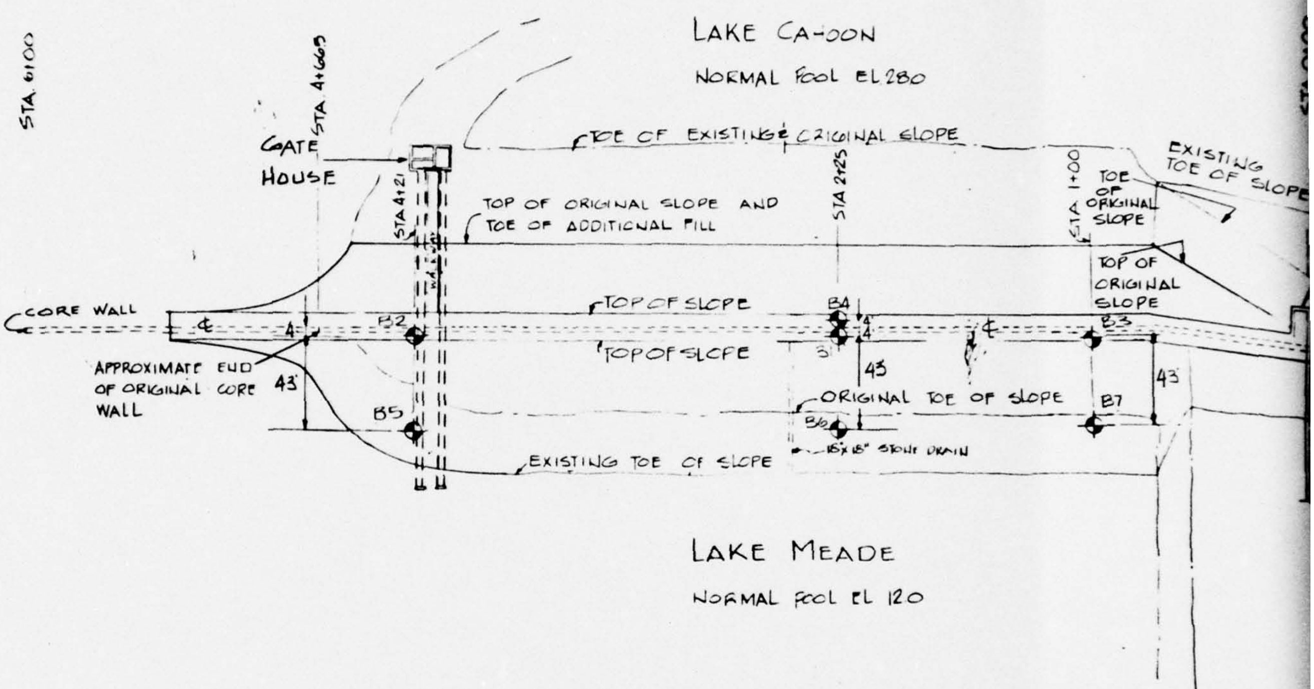
24" GATE VALVE WITH WOOD LODGED IN IT WHICH PREVENTED CLOSING

VIEW OF EMBANKMENT FROM GATE HOUSE LOOKING NORTH (Upstream Slope)

VIEW OF DETERIORATED WET WELL LADDER



VIEW INTO ONE OF WET WELLS OF 30" SLUICE GATES


LAKE CAHOON
NORMAL POOL EL 280
STA 6+00
STA 4+665
GATE HOUSE
STA 4+21
TOE OF EXISTING & ORIGINAL SLOPE
STA 2+25
STA 1+00
TOE OF ORIGINAL SLOPE
EXISTING TOE OF SLOPE
TOP OF ORIGINAL SLOPE AND TOE OF ADDITIONAL FILL
TOP OF ORIGINAL SLOPE
CORE WALL
TOP OF SLOPE
B4
B2
B3
TOP OF SLOPE
APPROXIMATE END OF ORIGINAL CORE WALL
43'
43
ORIGINAL TOE OF SLOPE
B5
B6
B7
18"x18" STONE DRAIN
EXISTING TOE OF SLOPE
LAKE MEADE
NORMAL POOL EL 120

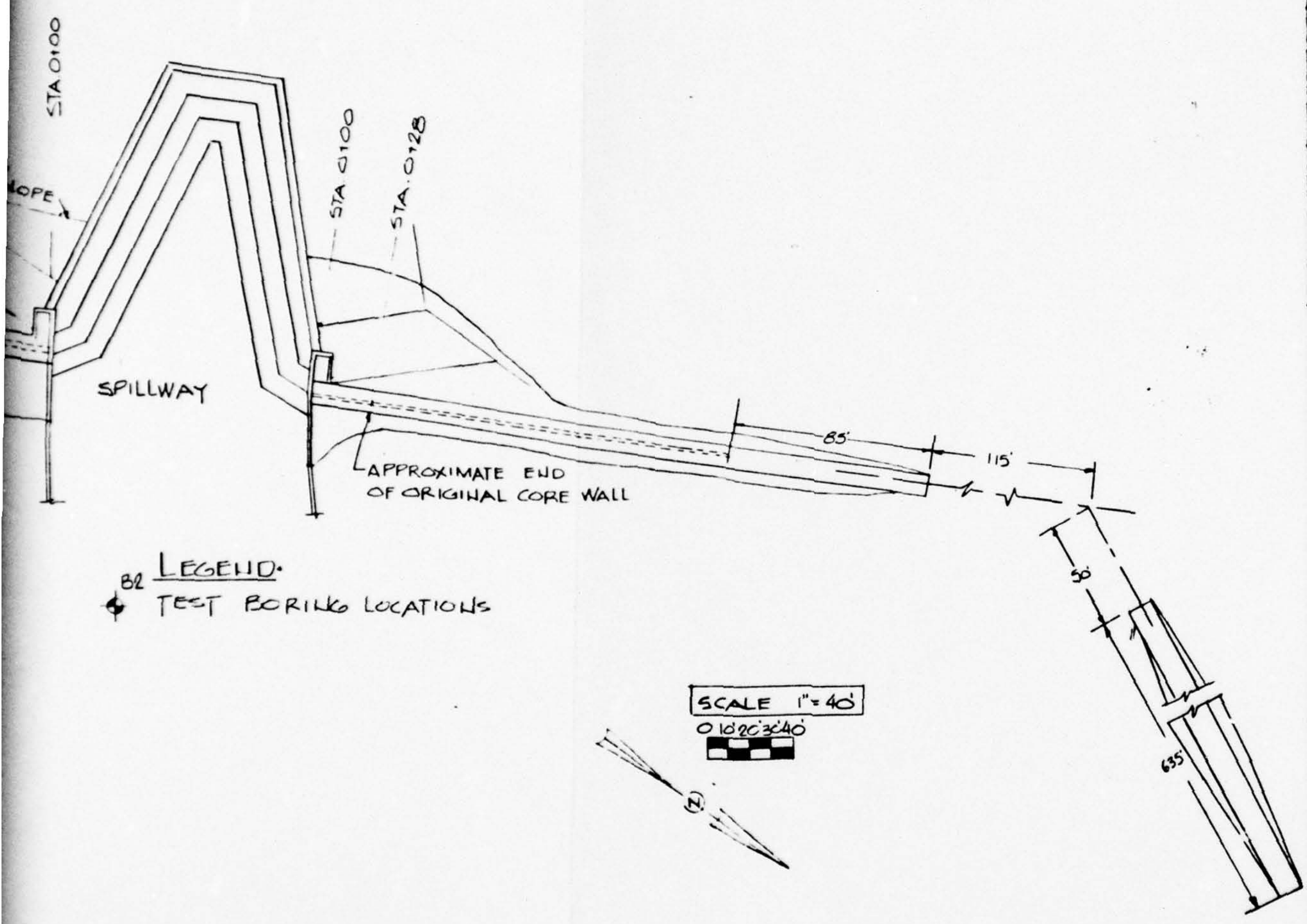
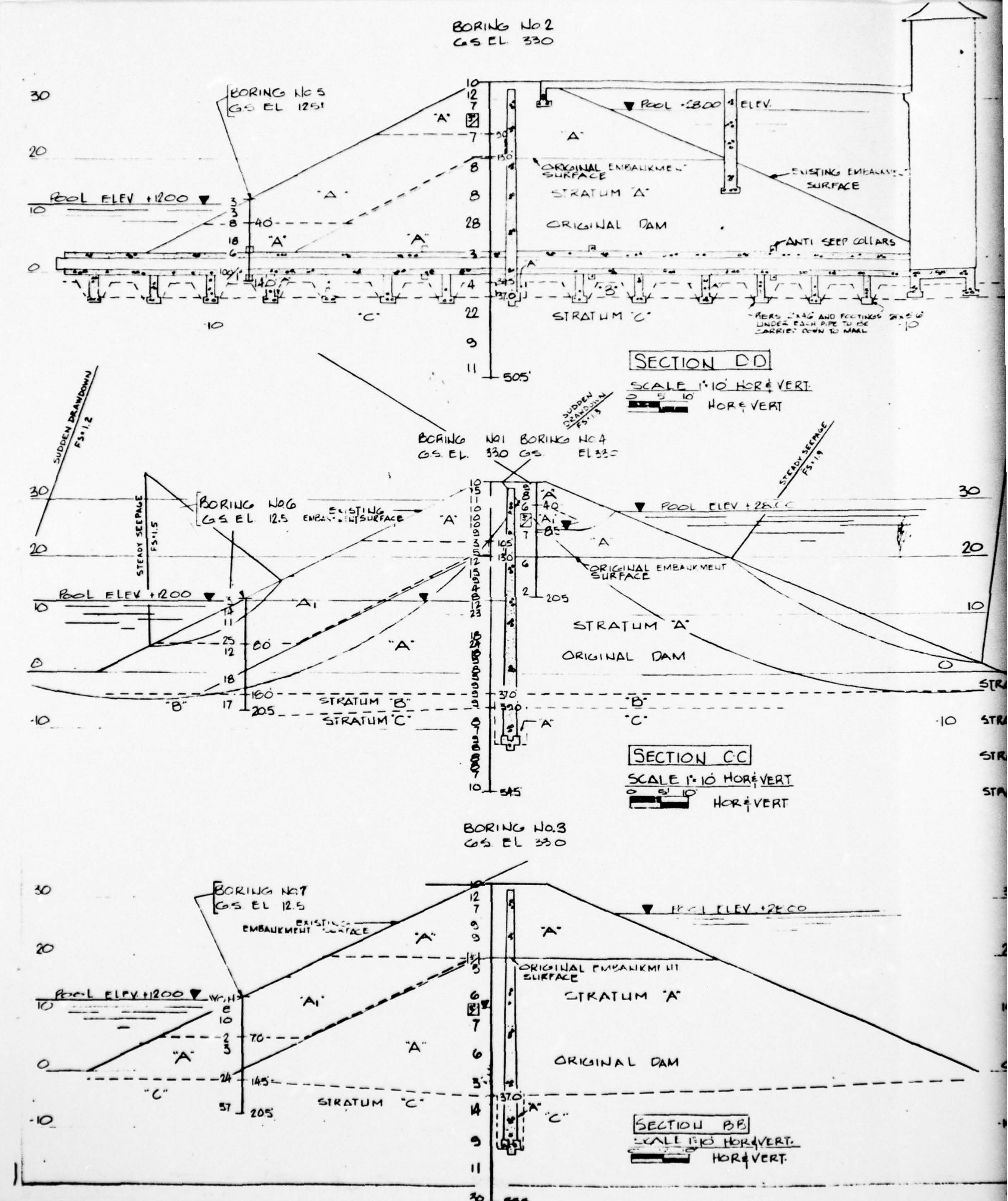


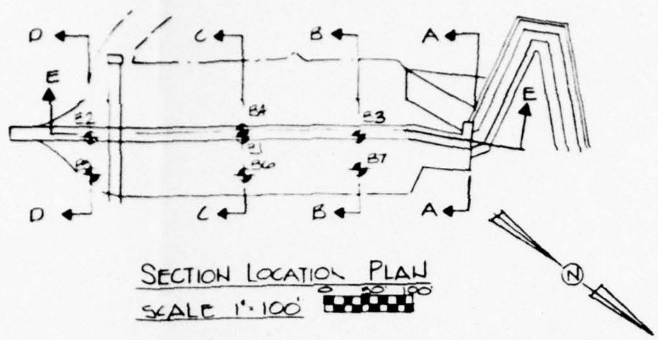
PLATE 1

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS, SOIL MECHANICS AND FOUNDATIONS
BETHESDA, MARYLAND . RICHMOND, VIRGINIA

TEST BORING LOCATION PLAN	SCALE 1"=40'	DATE 11/3/78
	DRAWN BY RSH	CK'D. BY
	DWG. NO.	SHEET NO. F1

2


BORING No 2
GS EL 330
BORING No 5
GS EL 12.5'
POOL ELEV +28.00 ELEV.
ORIGINAL EMBANKMENT SURFACE
EXISTING EMBANKMENT SURFACE
STRATUM "A"
POOL ELEV +12.00
ORIGINAL DAM
ANTI SEEP COLLARS
STRATUM "C"
PIERS 2'x4'6" AND FOOTINGS UNDER EACH PIPE TO BE CARRIED DOWN TO MARL
50.5'
SECTION DD
SCALE 1"=10' HOR & VERT
HOR & VERT
SUDDEN DRAWDOWN FS=1.2
SUDDEN DRAWDOWN FS=1.3
BORING No 1 BORING No 4
GS EL 33.0 GS EL 33.0
STEADY SEEPAGE FS=1.9
BORING No 6
GS EL 12.5
EXISTING EMBANKMENT SURFACE
POOL ELEV +28.00
STEADY SEEPAGE FS=1.5
ORIGINAL EMBANKMENT SURFACE
POOL ELEV +12.00
STRATUM "A"
ORIGINAL DAM
STRATUM "B"
STRATUM "C"
54.5'
SECTION CC
SCALE 1"=10' HOR & VERT
HOR & VERT
BORING No.3
GS EL 33.0
BORING No 7
GS EL 12.5
EXISTING EMBANKMENT SURFACE
POOL ELEV +28.00
ORIGINAL EMBANKMENT SURFACE
POOL ELEV +12.00
STRATUM "A"
ORIGINAL DAM
STRATUM "C"
SECTION BB
SCALE 1"=10' HOR & VERT
HOR & VERT



SECTION LOCATION PLAN
SCALE 1"=100'

GENERAL NOTES

1. NUMBERS TO THE LEFT OF THE BORING COLUMN INDICATE NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D., 1 3/8 INCH I.D. SAMPLING SPOON 12 INCHES USING A 140 LB. HAMMER FALLING 30 INCHES PER ASTM D-1586.
2. ESTIMATED GROUNDWATER LEVEL INDICATED BY ▼; THESE LEVELS ARE ONLY ESTIMATED FROM AVAILABLE DATA AND MAY VARY WITH PRECIPITATION, POROSITY OF SOIL, SITE TOPOGRAPHY, ETC.
3. KEY TO ABBREVIATIONS: G.S. = GROUND SURFACE ☒ = UNDISTURBED TUBE SAMPLE.
4. THIS DRAWING CONTAINS INTERPRETATION OF TEST BORINGS AND SHOULD NOT BE USED AS PART OF THE CONTRACT DOCUMENTS.
5. THESE PROFILES WERE DEVELOPED BETWEEN WIDELY SPACED BORINGS. ONLY AT THE BORIN LOCATIONS SHOULD THEY BE CONSIDERED AS AN APPROXIMATELY ACCURATE REPRESENTATIC AND THEN ONLY TO THE DEGREE IMPLIED BY THE NOTES ON THE BORING LOGS.
6. TEST BORINGS DRILLED BY AYERS AND AYERS, INC. RICHMOND, VIRGINIA AND INSPECTED BY SCHNABEL ENGINEERING ASSOCIATES.

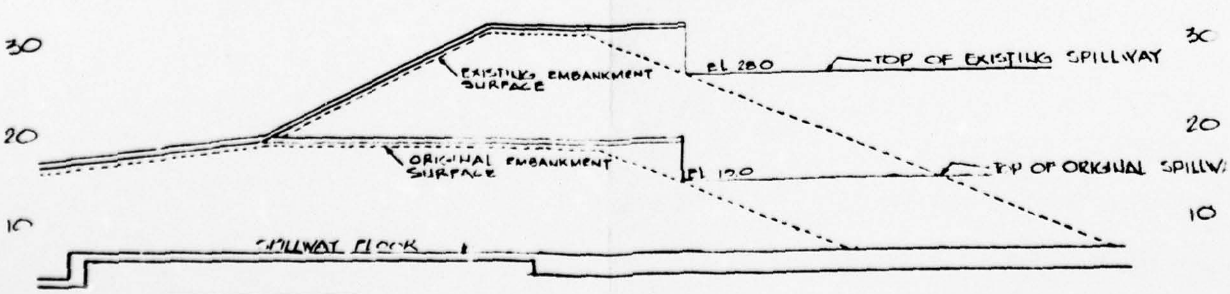
STRATA DESCRIPTIONS

STRATUM A: BROWN TO GRAY FINE TO COARSE SILTY TO CLAYEY SAND, FILL, (SM) TRACE ORGANIC MATTER AND GRAVE WITH SHELL FRAGMENTS; VERY LOOSE TO FIRM (N=2 TO 28).

STRATUM A1: BROWN TO GRAY FINE TO MEDIUM SILTY CLAYEY SAND (SC) AND SILTY CLAY (CL), FILL, TRACE ORGANIC MATTE GRAVEL AND SHELL FRAGMENTS; SOFT TO VERY STIFF CONSISTENCY (N=3 TO 25).

STRATUM B: BROWN GRAY TO GREEN FINE TO COARSE SAND (SM), TRACE SILT, ORGANIC MATTER AND GRAVEL; AND FINE CLAYEY SAND (SC); TRACE GRAVEL; LOOSE TO FIRM (N=4 TO 16).

STRATUM C: GRAY TO GREEN FINE TO COARSE CLAYEY TO SILTY CLAYEY SAND (SC), FINE SANDY CLAYEY SILT (ML) AND SILTY CLAY (CL), SOME FINE SAND, WITH SHELL FRAGMENTS; STIFF TO HARD CONSISTENCY (N=7 TO 57).

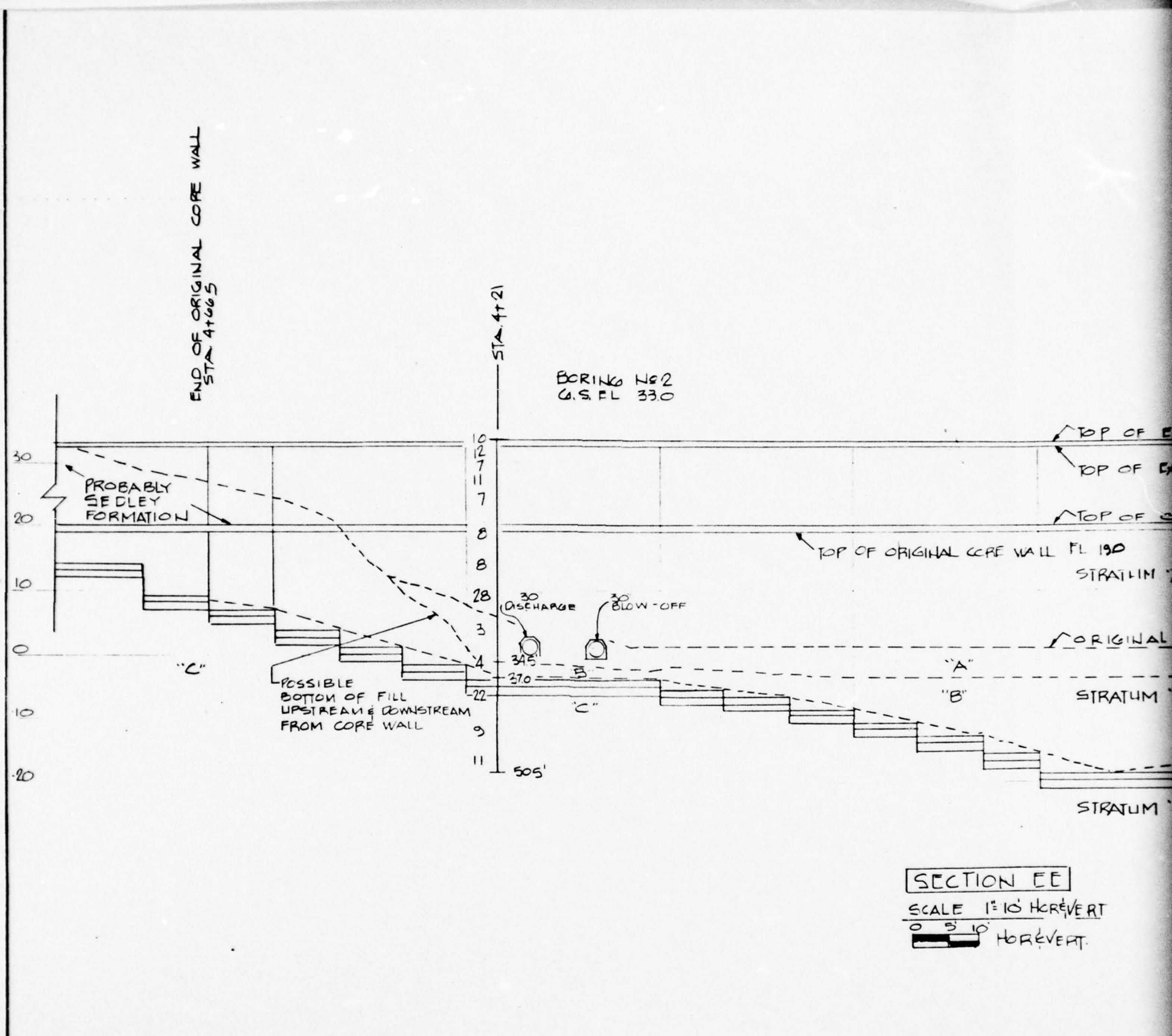


SECTION A-A
SCALE 1"=10' HOR. & VERT.
0 5 10
HOR. & VERT.

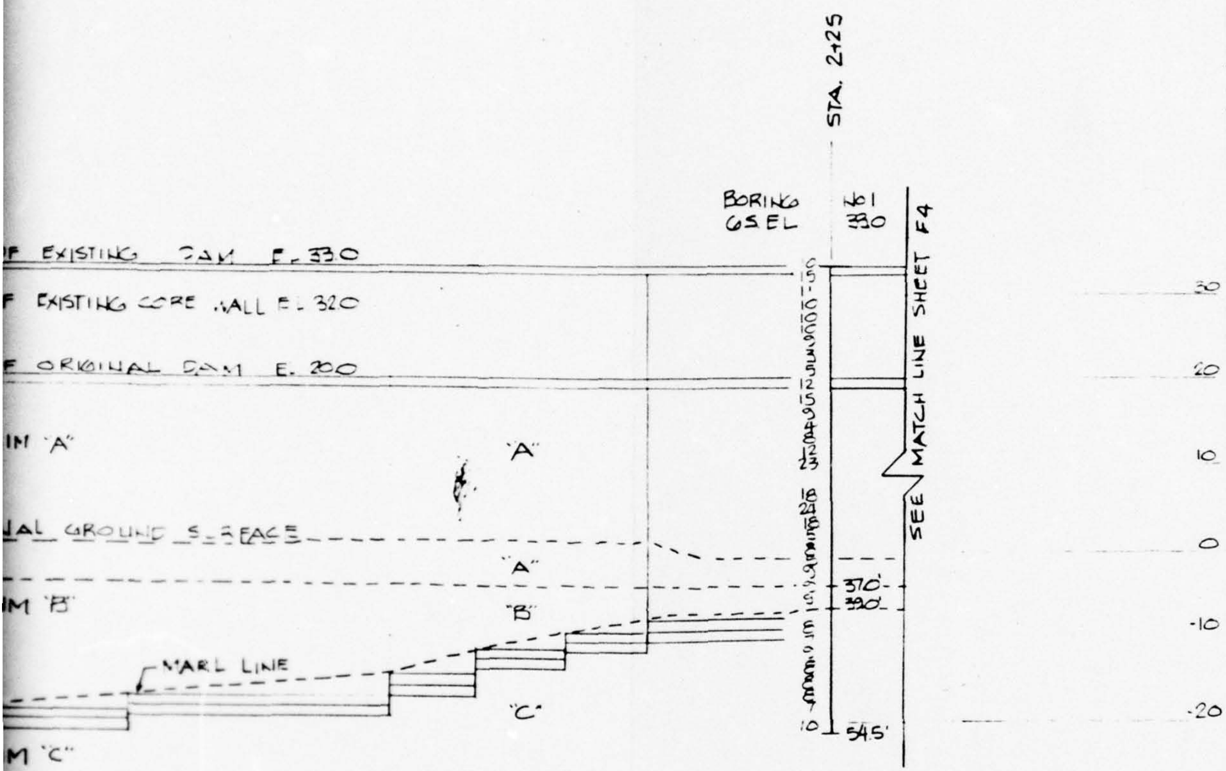
PLATE 2

SCHNABEL ENGINEERING ASSOCIATES CONSULTING ENGINEERS, SOIL MECHANICS AND FOUNDATIONS BETHESDA, MARYLAND - RICHMOND, VIRGINIA		
... CF ... WATER SUPPLY		
...MS-CAHOON ... VIRGINIA		
ESTIMATED SUBSURFACE PROFILES	SCALE AS SHOWN	DATE 11/3/7
	DRAWN BY K.P.H.	CK'D. BY [illegible]
	DWG. NO. V1F142	SHEET NO. F-2

2



END OF ORIGINAL CORE WALL
STA 4+66.5
STA 4+21
BORING No 2
G.S. EL 33.0
TOP OF E
TOP OF EX
TOP OF C
TOP OF ORIGINAL CORE WALL EL 19.0
STRATUM "A"
ORIGINAL
STRATUM "B"
STRATUM "C"
PROBABLY SEDLEY FORMATION
POSSIBLE BOTTOM OF FILL UPSTREAM & DOWNSTREAM FROM CORE WALL
30" DISCHARGE
30" BLOW-OFF
"A"
"B"
"C"
34.5
37.0
50.5'
30
20
10
0
-10
-20
10
12
7
11
7
8
8
28
3
4
22
9
11
SECTION EE
SCALE 1"=10' HOR & VERT
0 5 10
HOR & VERT.



NOTE. MARL LINE & CORE WALL FOUNDATION ELEVATIONS WERE TAKEN FROM ORIGINAL DRAWINGS AND ARE NOT NECESSARILY REPRESENTATIVE OF AS BUILT CONDITIONS.

FOR SECTION LOCATION SEE SHEET F2

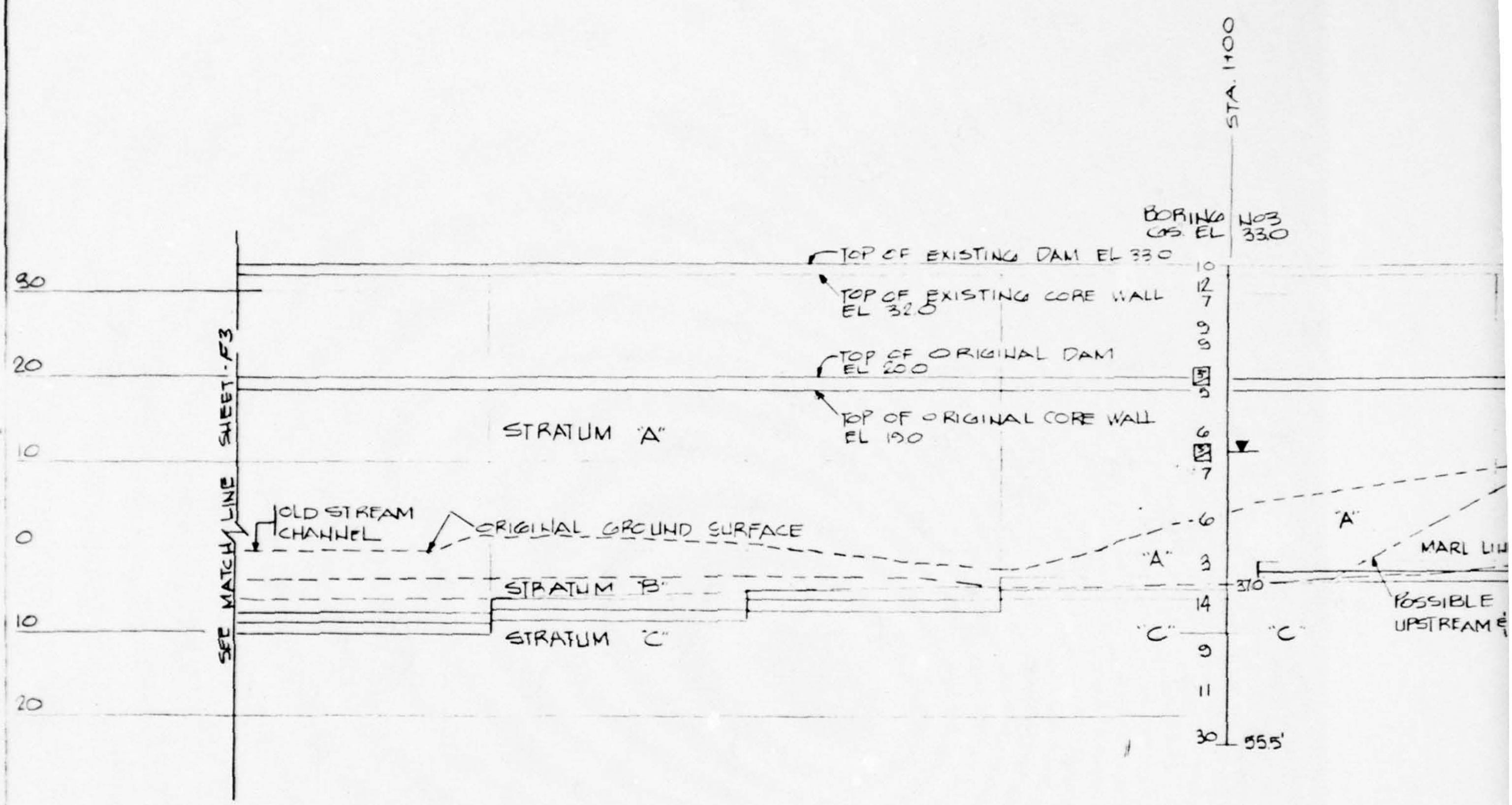
PLATE 3

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS, SOIL MECHANICS AND FOUNDATIONS
BETHESDA, MARYLAND · RICHMOND, VIRGINIA

CITY OF PORTSMOUTH WATER SUPPLY
DAVIS - CAHOON SUFFOLK VIRGINIA

ESTIMATED SUBSURFACE PROFILES	SCALE AS SHOWN	DATE
	DRAWN BY RBH	CKD. BY
	DWG. NO. V78142	SHEET NO F3

2



SECTION E-E

SCALE 1"=10' HOR & VERT.

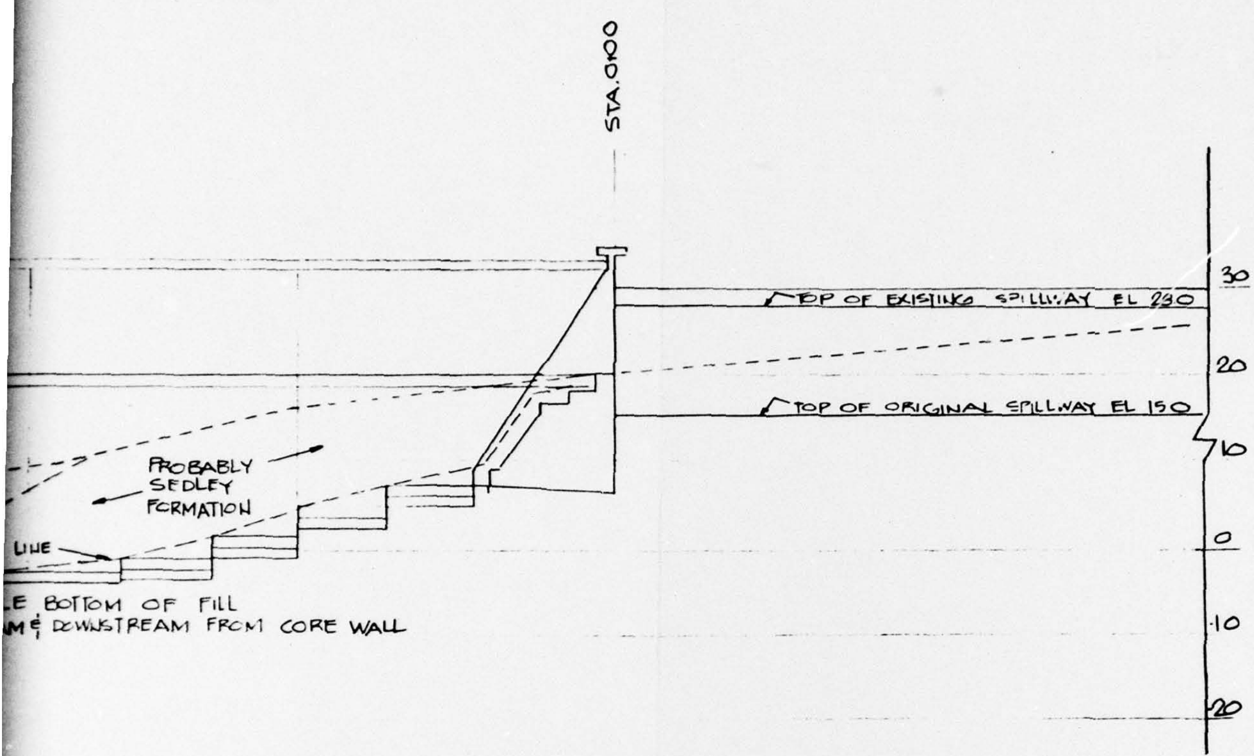
0 5 10 HOR & VERT

AD-A075 317 MARTIN (DEWARD M) AND ASSOCIATES INC WILLIAMSBURG VA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. LAKE COHOON DAM (INVENTORY NUMBER --ETC(U)
AUG 79 P SEILE DACW65-78-D-0015

UNCLASSIFIED NL

2 OF 2
AD
A075317

END
DATE
FILMED
11-79
DDC



NOTE. MASONRY LINE & CORE WALL FOUNDATION ELEVATIONS WERE TAKEN FROM ORIGINAL DRAWINGS AND ARE NOT NECESSARILY REPRESENTATIVE OF AS BUILT CONDITIONS.

FOR SECTION LOCATION SEE SHEET F2

PLATE 4

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS SOIL MECHANICS AND FOUNDATIONS
BETHESDA, MARYLAND · RICHMOND, VIRGINIA

[illegible] COUNTY
[illegible], VIRGINIA

ESTIMATED SUBSURFACE PROFILES	SCALE [illegible]	DATE 11/3/78
	DRAWN BY [illegible] BH	CK'D BY [illegible]
	DWG. NO [illegible]	SHEET NO F4

2

APPENDIX V

REFERENCES

LIST OF REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, Department of the Army, Office of the Chief of Engineers, Washington, D.C. 20314

2. Inspection Report, Lake Cohoon Dam, for Portsmouth Water Reservoirs, Portsmouth Utilities Department, Suffolk, Virginia, by J. K. Timmons & Associates, Inc., Richmond, Virginia.